CIVIL ENGINEERING

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SIMS BRIDGE OVER THE SACRAMENTO RIVER, CALIFORNIA
Built by Civilian Conservation Corps Boys

Volume 4 AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852 Number 7

JULY 1934

Among Our Writers

E. W. KRAMER is Regional Engineer for the California Region of the U. S. Forest Service and has charge of the engineering phases of the work on the National Forests of California. He is also the representative of the Federal Power Commission on all projects located in the National Forests in California. He served as a member of the California Power Board and represented the former Executive Secretary of the Federal Power Commission on the Federal-State Commission appointed by President Hoover and Governor Young to study the State Water Plan.

ALBERT A. MITTAG graduated from the Cooper Union Institute of Technology in 1925 with the degree of B.S. in Civil Engineering and received his C.E. from the same institution in 1932. Since 1925 he has been in Salvador and Colombia on highway location and construction. From 1930 to 1933 he was engineer, assistant engineer, and chief engineer for $2^1/_2$ years with the Constructing Quartermaster at Albrook Field, Canal Zone. At present he is with the Constructing Quartermaster at Langley Field, Va.

CLARENCE J. VELZ has specialized in sanitary and public health engineering. He assisted in making a study of, and report on, the pollution of New York Harbor for the New York State Board of Health. At present Mr. Velz is connected with the Federal Emergency Administration of Public Works in New Jersey, working under the State Engineer.

E. L. CHANDLER, after receiving the degree of B.S. in Civil Engineering at Brown University, had experience in a variety of municipal work in New London and Groton, Conn. From 1915 to 1918 he was assistant engineer with the Board of Water Supply of Providence, R.I. From 1918 to 1923 he served as assistant division engineer with the Miami Conservancy District on flood prevention work. Since 1923 he has been associated with the Price Brothers Company, of Dayton, Ohio, general contractors, and has specialized in the construction of hydroelectric developments.

ALFRED BRAHDY has been engaged on New York City rapid transit work in various capacities for 27 years, and for the past 8 years has been designing engineer with the Board of Transportation on the planning of the city-owned Independent Subway System. From 1930 to 1932 he was the resident representative in Chicago for Robert Ridgway on the planning of the proposed State Street Subway.

BUNKICHI OKAZAKI received his education at Sapporo College in Japan. For 25 years he was in the service of the Hokkaidocho Government on public works, including the improvement of the Ishikari River. More recently his experience in river control has been utilized in Manchukuo.

LAMOTTE GROVER received his B.S. in C.E. at Kansas State College in 1924. He has been in charge of bridge design for the State Highway Commission of Kansas since 1929, when the design department was first organized. He started to work for the Commission in 1921 as a concrete pavement inspector and has been with the organization continuously except for two brief periods. He has served as draftsman, as designer, and as resident engineer in charge of road, bridge, highway-pavement, and city-pavement construction. The Bridge Department of the Kansas Highway Commission has designed a number of unusual and interesting bridges during the last three years.

THOMAS J. NOLAND, JR., in 1929 received his B.S. in Engineering from the California Institute of Technology and in 1931 his M.S. in Civil Engineering. He was then appointed a junior engineer in the U. S. Reclamation Bureau, and was assigned to the Engineering Materials Laboratory of the University of California, where he worked for two years on the cement and concrete research program for Boulder Dam. When this work was completed, in the fall of 1933, he was transferred to the Denver Office.

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NUMBER 7

Engineering Activities of CCC Camps in California

Chief Phases Include Bridge Building, Road Construction, and Control of Floods and Erosion

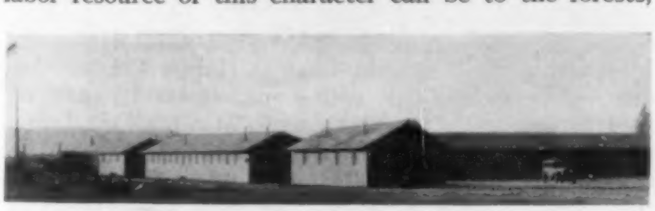
By E. W. KRAMER
MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
REGIONAL ENGINEER, U. S. FOREST SERVICE, SAN FRANCISCO, CALIF.

THE 150 camps of the Civilian Conservation Corps established in California during the first winter have accomplished a tremendous amount of useful work in the forests and adjacent areas. The Corps has passed through the second six months' period of its existence. In addition to the construction of many miles of truck trails and telephone lines and pest control work and forest thinning on thousands of acres, the 30,000 men enrolled in these camps are active on numerous projects of a purely engineering character. They have built 18 excellent highway bridges, have made relief maps of the National Forests in the state, constructed along the banks of vagrant rivers miles of sand levees protected by brush-filled wire fences, and covered many acres with the necessary ditches and control works to spread the flood waters of flashy southern streams over adjacent gravel beds in order to return to ground water flows that otherwise would waste to the ocean. Also, six small dams have been built to create reservoirs for the collection of silt and for the measurement of run-off. These CCC camps continue to serve two useful purposes—the rehabilitation of the Nation's young men and the improvement of the forest areas.

PRIMARILY the object of the Civilian Conservation Corps program is to relieve unemployment, particularly among boys just finishing school, to get them off the streets, and prevent them from adopting shiftless ways at a very important time in their lives. It is obvious that a program of this kind will be of great benefit throughout the country—to the boys themselves, to the forests, and to the locality in which the camps are located.

THE FORESTERS' PROBLEM

In order to understand the great benefit that a large labor resource of this character can be to the forests,



CCC CAMP NO. 1624, SAN JACINTO, CALIF.

some of the foresters' problems will be described briefly. The area of the National Forests in the United States is 162 million acres, of which 24 million acres are in California. The principal problems in connection with this area, other than those concerned with the marketing of forest products, are fire prevention, protection from insect pests and vegetable diseases, control of floods and erosion, opening up of the area for public use, and development of recreational sites. The problems in the National Parks, state forests, and certain localities near the National Forests are to a certain extent similar.

To solve these problems, there have been established in the whole United States 1,500 CCC camps containing a total of 300,000 men and costing approximately 250 million dollars per year, or $167,000 per year per camp. During the winter of 1933–1934, there were 150 of these camps in California, employing a total of 30,000 men and costing the Government annually about 25 million dollars.

TABLE I. WORK ACCOMPLISHED BY CCC CAMPS IN FIRST SIX MONTHS

IN THE UNITED STATES

10,000 miles of truck trails	1,700,000 acres on which tree and plant disease control is completed
5,000 miles of telephone lines	3,500,000 acres on which erosion control is completed
4,000 miles of fire breaks	205,000 acres on which the stand of timber has been thinned
6,600 miles of roadside fire protection work	388,000 acres on which erosion has been controlled
1,700 lookout towers	21,000 acres of revegetative work completed
4,300 bridges constructed	26,000 acres reforested by planting
68,000 dams constructed for erosion control	
800,000 areas protected from insect pests	

IN CALIFORNIA

925 miles of truck trails	212 miles of fire breaks
593 miles of telephone lines	976 miles of road protected from fire

In addition, work has been conducted to control blister rust, the pine beetle, and other insects; and the construction of drift fences, bridges, levees, flood control structures, and dams has been accomplished.

A Through-Truss Span of 75 Ft Over Stuart's Fork of the Trinity River

Constructing a Bridge Over the Sacramento River at Delta, with Main Span of 98 Ft

300-Ft Suspension Span Over the Klamath River at Happy Camp

Deck Truss 75 Ft Long Across the North Fork of Trinity River

Sims Bridge, a 160-Ft Suspension Span, Over the Sacramento

CALIFORNIA BRIDGES ERECTED BY CCC BOYS

To show roughly the work accomplished in the United States and in California during the first six months' period after the camps were established, the statistics in Table I are presented.

BRIDGE CONSTRUCTION

In California the engineering work of the CCC boys includes the construction of bridges, relief mapping of the forests, flood control projects such as water spreading, the building of six Ambursen-type dams for experimental purposes, and a large amount of work to control erosion. In this state there are six bridge crews, each attached to a CCC camp. Each crew is under a construction engineer in charge of from 5 to 7 foremen, who are specialists, as follows: one carpenter, one blacksmith, one steel erector, one rigger, and one concrete technician. These men direct the work of from 30 to 50 CCC boys on each bridge. The bridges are designed according to Government 15-ton loading specifications, which provide for a string of 15-ton trucks following each other at a reasonable interval.

The largest bridge yet completed is a 300-ft suspension span across the Klamath River at Happy Camp. Another suspension bridge, with a span of 160 ft, has been constructed across the Sacramento at Sims. At Delta, also across the Sacramento River, a bridge with a main span of 98 ft has been built. Across Stuart's Fork of the Trinity River a bridge has been constructed of the through-truss type with a main span of 75 ft. Of the same length is the main span of the deck-truss bridge that has been completed across the North Fork of the Trinity River. Altogether, some 18 bridges, costing from $5,000 to $25,000 each, have been built in California by the Corps.

At Castella a garage has been converted into a drafting office for a crew of 30 CCC men who are engaged in constructing relief maps of the National Forests of California. The maps are on a scale of 1 in. to the mile and

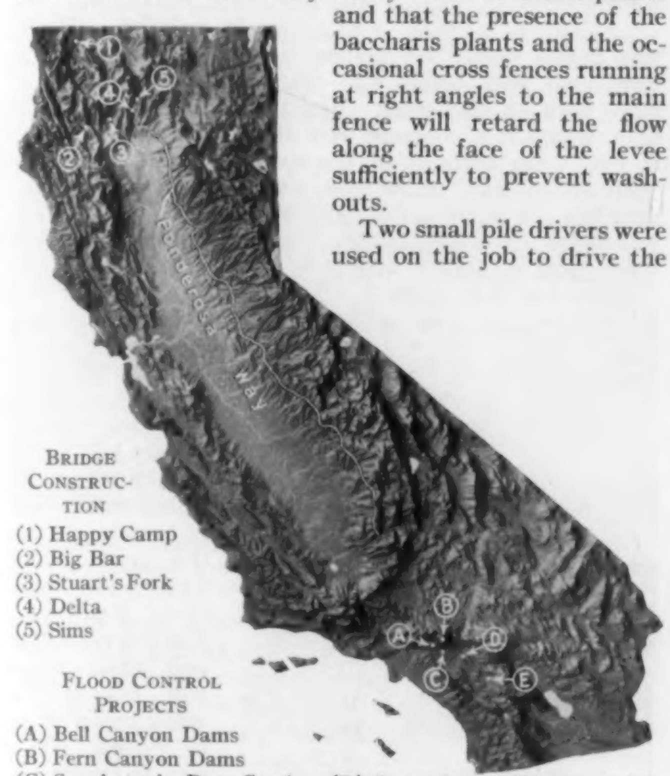
a vertical scale four times the horizontal. These maps will be of immense value in all administrative work pertaining to the forests, particularly in establishing lookout points. By placing small electric lights on the various peaks and photographing the relief model with these lights burning, the area that can be seen from each lookout point is easily and accurately shown.

BUILDING AND PLANTING LEVEES

In the past winter there were four flood-control projects under way, all in southern California. One consists of controlling floods on the San Jacinto River and its principal tributary, Bautista Creek. From October 1933 to April 1934 a CCC camp was engaged in this work, which consisted of constructing four miles of levee, 6 ft high and about 5 ft wide on the crest, by means of a $1^1/_4$-yd drag-line excavator. The material of which the levee was made is but little better than ordinary sand. Between the levee and the stream, and about 10 ft from the foot of the levee, there are two rows of woven wire fencing about 5 ft apart, attached to posts consisting of boiler tubing 16 ft long driven 10 ft into the ground. The berm between the fence and the levee is 10 ft wide.

On the water side of the levee the berm was planted with various types of vegetative cover that are adapted to desert conditions. The principal plant used is baccharis, which looks very much like willow but is better able to withstand drought. Wild tobacco and salt brush have also been planted. It is hoped to get the levee covered with a fairly heavy stand of desert plants and that the presence of the baccharis plants and the occasional cross fences running at right angles to the main fence will retard the flow along the face of the levee sufficiently to prevent washouts.

Two small pile drivers were used on the job to drive the boiler tubing. The Forest Service rents a drag-line machine for \$5 an hour, the owner making all repairs and furnishing an operator. The Forest Service furnishes gas and oil and one man who is called a "swamper." Over a period of six months the cost of the drag-line amounts to about \$8,000, whereas the cost of the camp other than the drag-line is about \$90,000.



Relief Map of California, with Location of Some CCC Engineering Activities

This drag-line moves about 750 yd in an 8-hr run, which represents a cost for building the levee, not including planting and fencing, of about 8 cents per cu yd of levee.

SANTA ANA RIVER SPREADING GROUNDS

On the flood control project on the Santa Ana River, use is being made of the diversion dam, canal, and part of the spreading works previously built by the Tri-County District, consisting of Orange, Riverside, and San Bernardino counties. The main diversion dam on the river is about at the point where the river debouches on the plain near San Bernardino.

San Jacinto River Levee, Built of Sand, Planted to Baccharis, and Protected by Brush Packed Between Two Rows of Fencing

According to the original plan, delivery of the water from the canal to the spreading area was accomplished by constructing a series of earth dams at right angles to the flow of the river and in the flood plain immediately adjacent to the main stream. These dams varied in height from 3 ft to 20 ft in some places. The material was placed on very steep slopes, and nearly all the dams failed because water broke through them in series at points where they crossed small gulches. In the new project the spreading area lies along the main channel of the river, downstream from the original spreading area. The feeder canal is some 12 ft wide on the bottom and 4 ft deep. The lateral canals, located at right angles to the feeder canal at intervals of 500 ft, are 8 ft wide on the bottom and 2 ft deep, and vary in length, averaging about 2,000 ft. At intervals of about 100 ft along the laterals, 12-in. pipes were laid through the ditch banks in order to distribute the water over the spreading area.

Work on this project, begun in April 1933, was performed entirely by hand for the first six months. It absorbed all the labor of a 200-man camp for this period. During this time, 900 ft of main canal and 7,000 ft of lateral were constructed. No diversion structures were built and no concrete pipes were placed. It was estimated that the average amount of excavation per man per day was about one yard, making the excavation cost approximately \$2.50 per cu yd. At the beginning of the second six months' period, a $1^1/_4$-yd shovel was rented to excavate the main canal and laterals. The rent of the shovel was \$3.50 per hr, with operator, the owner to assume all burden of upkeep and repair. In a period of six months, this shovel cost a total of about \$6,000, or about \$30 per day, including gas and oil provided by the Government. It moves about 300 yd in an 8-hr shift, so that the cost per cubic yard is now about 10 cents instead of \$2.50. In the first three months of the second six months' period, there had been constructed 1,000 ft of main canal, 17,000 ft of laterals, and a large number of concrete diversion structures, one at the intake of

BAUTISTA WASH, SHOWING FENCE READY TO RECEIVE BRUSH

CROSS FENCING AND PLANTING TO REDUCE VELOCITY OF CURRENT

BOYS PLANTING BACCHARIS

each lateral. The placing of the 12-in. pipes was also well under way at the end of this three months' period.

In the second period the men were engaged in building the concrete structures, trimming the banks, and placing the 12-in. concrete pipes. They were more satisfied with their accomplishment than before the shovel was put on the job. With the aid of the shovel, at the end of the second six months' period these men had constructed a spreading project on the Santa Ana capable of getting the water into the gravel cone at the rate of 600 cu ft per sec.

SPREADING GROUNDS ON DEER AND SAN ANTONIO CREEKS

There are two other flood control projects under way, one on San Antonio Creek and one on Deer Creek. These streams flow southwesterly from the Sierra Madre Mountains. San Antonio Creek leaves the mountains opposite Pomona and Deer Creek, near Ontario. On the latter the spreading works consist of a concrete diversion structure at a point on the stream where it leaves the mountains. Here its cone is very steep, the average grade being about 10 per cent. The water is diverted into a feeder canal which supplies ditches built on contours. Because of the steepness of the country and the character of the material, almost all of which is large gravel, it was necessary to build a wire-bound rock wall for the lower embankment of each ditch. These walls are about 3 ft high and 2 ft wide on top. The water is run from the main feeder canal into the ditches, whence it is turned out at intervals of about 50 ft down the cone.

Deer Creek has a drainage area of about $3^1/_2$ sq miles, and the spreading works have a capacity of about 150 cu ft per sec. It is possible that maximum floods for short periods may be considerably greater than the capacity of the spreading works, in which case the water will go over the concrete diversion structure and down the channel. This will be a rare occurrence. A canal 18 ft wide and 2 ft deep was dug below the spreading area to lead any stray water back into Deer Creek and thus protect the citrus groves lower down from any damage that might occur from careless operation of the spreading works.

The area below the spreading works, which is dependent on water pumped from the Deer Creek cone, is entirely planted with oranges, lemons, and other citrus fruits. The Deer Creek water that is spread will be a valuable addition to the supply. The ground-water level in the wells in this area has been getting deeper, as is the case in many places elsewhere in California.

It is estimated that the construction of these spreading works will require 12 months. Work was begun about November 1, 1933, and should be completed well before the first of October 1934. A large part of the work consists of constructing the dam, which is a concrete weir about 8 ft high and 72 ft long. On account of the steep slope of the stream both above and below the weir, and because large boulders are carried downstream in flood periods, it was found necessary to line the rollway of the dam throughout its length with steel rails.

Work was started on the San Antonio project in October 1933, and it is expected that it will be completed in September 1934. This project differs from the Deer Creek project in that the area on which the works are located is much flatter, the grade being probably only about 2 per cent. The diversion dam and main canal will deliver the water into a series of ponds, which are formed by check dams built on contours. These dams are about $5^1/_2$ ft high and are built of rock mattresses bound together with wire. The mattresses, which are about 2 ft thick, lie one on top of the other. Each check dam will back the water to a depth of 1 ft on the dam next above. It is estimated that water thus ponded will sink at the rate of at least 3 acre-ft per day, or $1^1/_2$ cu ft per sec per acre. The capacity of the spreading works is estimated roughly at 100 cu ft per sec.

DRIVING 16-FT BOILER TUBING FOR FENCE POSTS

It is the present policy to confine the activities of the CCC to work more directly connected with forestry or erosion control problems than are water conservation and river rectification projects.

RESERVOIRS FOR SILT AND RUN-OFF MEASUREMENT

Recently six reservoirs have been completed in the Angeles National Forest, below areas which have been set aside for experimental purposes. The initial work consisted of constructing six dams of the Ambursen type, each about 15 ft high and 30 ft long, capable of impounding about 10,000 cu ft of water, the storage space to be concrete lined. Immediately above each reservoir there is to be installed a concrete water-measuring device consisting of a Parshall flume and V-notch weir. The immediate object of the dams, flumes, and weirs is to measure the eroded material and run-off from these watersheds, the aim being to determine the effect of the vegetative cover on run-off and erosion.

Three of these dams are on the upper reaches of Bell Canyon at an approximate elevation of 2,500 ft and have been designated as Bell Canyon dams Nos. 1, 2, and 3. Their tributary drainage areas are 80.12, 103.98, and 65.18 acres, respectively. Bell Canyon is in the watershed of Big Dalton Canyon, one of the canyons draining the southern slopes of the San Gabriel Mountains of southern California.

According to present plans, data will be obtained on each of the drainage areas in its natural condition for a five-year period; then one of the watersheds will be burned off and maintained in a barren state; a second will be burned off and allowed to go back to its natural state; and the third will be maintained with the present cover. These three drainage areas are a part of a larger experiment which covers observations on larger drainage areas in this experimental forest. The other three dams to be used in conducting a similar experiment are located on Fern Canyon at about elevation 4,500 ft.

Cucamonga Spreading Ground on San Antonio Creek, for Returning Flood Flows to Ground Water

It is expected that the Parshall flumes will accurately measure stream flows of from 3 to 50 cu ft per sec. The arrangement is such that flows of less than 3 cu ft per sec will be measured by a V-notch weir. The silt washed down by the streams will be measured in the concrete-lined reservoirs. Both the Parshall flumes and the V-notch weirs will be supplied with recording gages.

PONDEROSA WAY, FOR FIRE PROTECTION

With CCC labor the U. S. Forest Service is now constructing what is called the Ponderosa Way, a road largely for fire protection purposes, which is located along the western slope of the Sierra Nevadas at an average elevation of 2,500 ft. It will be about 600 miles long, extending from a point opposite Redding to a point opposite Bakersfield. In many places the Ponderosa Way crosses bridges already constructed, but it will be necessary to build bridges of considerable size across the Middle Fork of the Yuba River, the Bear River, the Middle Fork of the American River, the Mokelumne River, and probably some of the other large streams draining the western slope of the Sierras. Plans have been prepared for these bridges, and their construction is now under way.

Concrete-Lined Experimental Reservoir for Measuring the Silt Content of Water

Contours Painted on Sides

ORGANIZATION

It has been found that, at least at the start, boys from agricultural areas, or from non-urban areas, are more able and more willing to work than those from large cities. This is true whether the cities are in the East or the West. However, the food and exercise connected with the work gradually build up the boys to a point where they are able to work, and in nearly every case they are willing to do so when properly handled. It has been found that they are particularly interested in work when it appears that they are accomplishing something. The Army's part is to construct the camps, enroll and feed the boys, look after their health and recreation, and maintain discipline.

The actual work which the men perform is planned, supervised, and directed by the U. S. Forest Service, the U. S. Park Service, or the State Forester. The Regional Forester in California, as the representative of Robert Fechner, Director of Emergency Conservation Work, passes on the suitability and relative importance of the various projects and recommends the location of the camps, both in state and National Forests and on flood control projects adjacent to the forests.

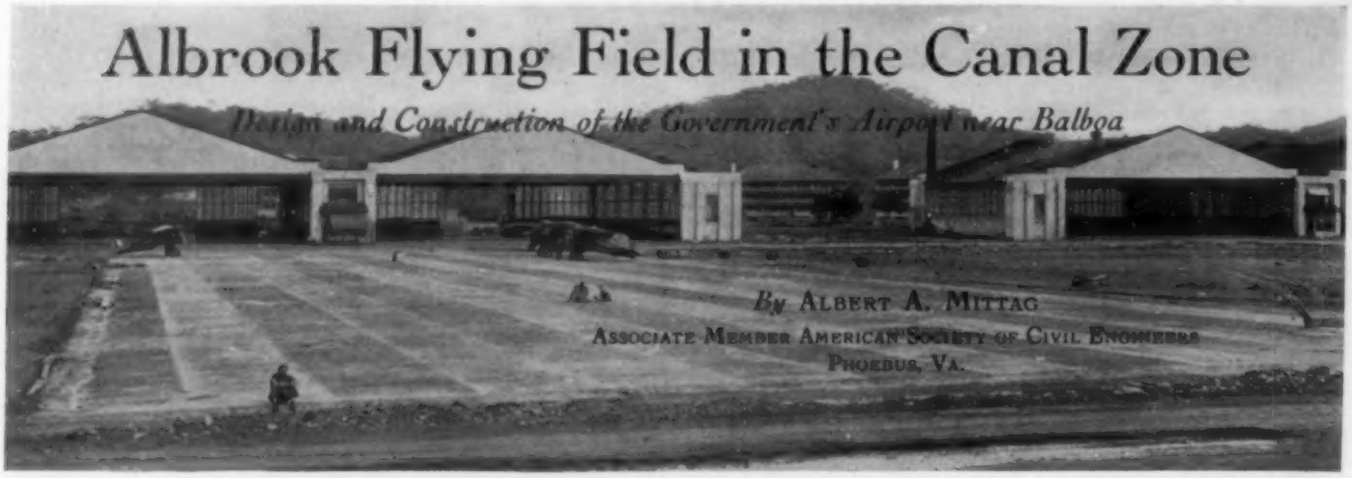
At the camp sites the Forest Service organization consists of a camp superintendent, a number of foremen, and in some cases specialists in the line of work being undertaken. A large force of technicians has been employed to plan and inspect certain classes of work, such as the control of beetles and blister rust on pine trees. On bridge jobs a construction engineer is in charge, and under him are foreman skilled in concrete and steel construction. Also, a small percentage of the enrolled men are paid from $10 to $15 a month extra as leaders and are used as straw bosses and often as foremen.

In many cases the camp superintendents are civil engineers of very high grade, who are willing to take these jobs because they cannot get better ones during the depression. The Forest Service was fortunate in getting high-type men for all supervisory positions.

Albrook Flying Field in the Canal Zone

Design and Construction of the Government's Airport near Balboa

By Albert A. Mittag
Associate Member American Society of Civil Engineers
Phoebus, Va.



Concrete Apron and Hangar No. 3

Many problems confronted the Government in its effort to build an adequate flying field in the Canal Zone and provide it with the necessary facilities. The site chosen was Albrook Field, two miles from Balboa, where a temporary field had been in use during the World War. Dredgings from the Canal prism, topped with dry fill, were utilized to build up the field. Two small rivers had to be placed in culverts beneath the area, and the Gaillard Highway, which originally traversed it, had to be relocated. Careful subdrainage was provided so that the field may be used within an hour after any tropical storm. The up-to-date hangars and aprons and the landscaped area on the adjacent hilly ground, where one- and two-family houses and other quarters have been built for the officers and enlisted personnel, are also described in this article by Mr. Mittag.

What is probably the largest, and soon to be the most complete airport south of Texas is slowly assuming shape at Albrook Field in the Canal Zone. It is evident that its location, only two miles from the Pacific mouth of the Panama Canal, is strategic even in this day of airplane speeds of 200 miles an hour. At this point the distance across the Isthmus is only about 45 miles as the crow flies. The need for the field was apparent during the hectic days of the World War, but it was not until 1925 that Congress authorized the necessary funds.

A careful perusal of an old map of the Panama Railroad shows that the present site of Albrook Field was an estuary of the Rio Grande and formed part of the Rio Grande Swamps. Into this estuary also flowed the Rio Maria Sala and the Rio Curundu. At high tides it was a lake, infested with alligators. Originally the main line of the Panama Railroad crossed this swamp, but during the construction of the Canal the railroad was moved to the south, to its present location. The old railroad bed from Corozal to Ancon was then converted into the Gaillard Highway, which up to 1932 bisected the Albrook Flying Field and reservation. During the construction of the Canal, considerable dirt, both dry and hydraulic, was dumped into this area and formed the first fill on the future field. When the prospective field was cleared of its vegetation the old narrow-gage rails and the waste dumps were found.

The preliminary field, built where the center of the new field now is, was made by putting a 3 to 4-ft layer of dry fill over material placed hydraulically. This served its purpose until 1929, when the first contract was let for a hydraulic fill varying in depth from 2 to 6 ft, covering part of the new field. The contractor for this work, The Panama Canal, obtained the fill from the Canal prism and pumped it to the site. Dredging operations presented no particular difficulties other than that of providing for the return flow. The field was divided into sections surrounded by dikes, with weirs constructed in numerous locations. The difficulty with the return flow was that much of the silt pumped to the sections remained in suspension and was carried back into the Canal, thus necessitating additional dredging to clear the Canal. The amount of dredged material totaled about a million cubic yards.

On completion of the hydraulic fill, a contract for 500,000 cu yd of dry fill was

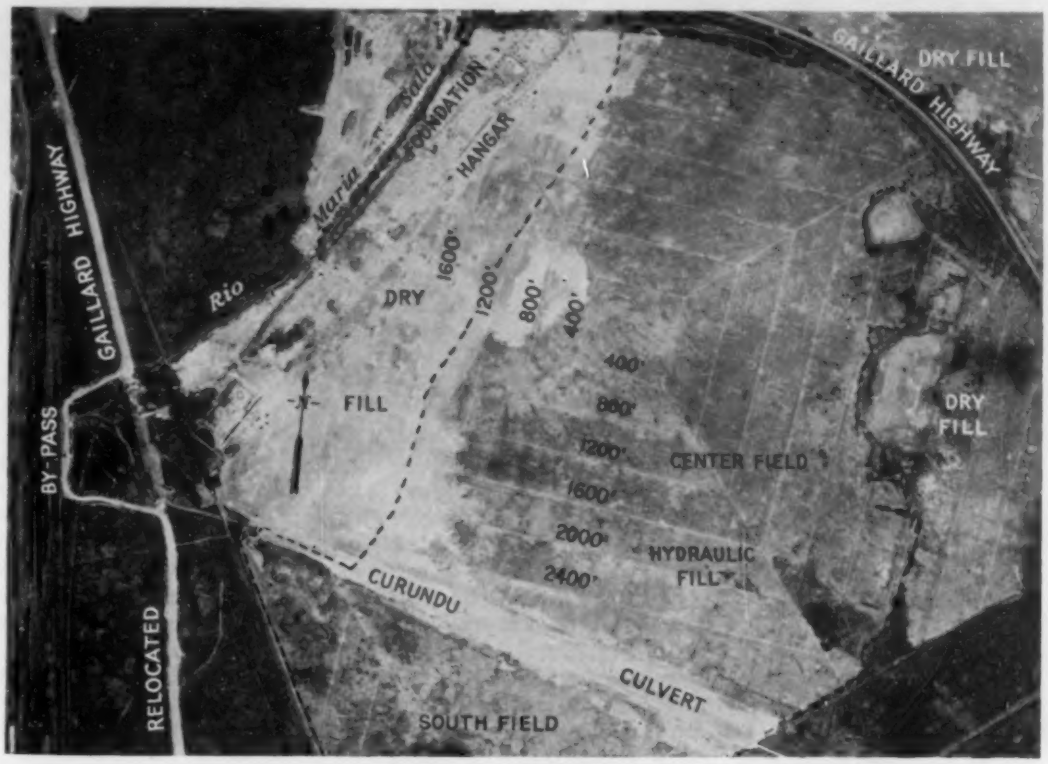
Albrook Field, near Balboa, C.Z., Looking South

let to build up the areas to the east and west of the hydraulic fill. It was the original intent of the plan to cover the whole flying field with one foot of dry fill, but on the advice of Professor Terzaghi, that the hydraulic fill would meet all requirements, this plan was abandoned. However, it was decided to place the dry fill in the area to the west to provide solid foundations for the warming-up aprons, roads, hangars, and shops, and in that to the east to provide a runway for bombers. This contract, begun in the early part of 1930, was completed a year later.

Through the southern part of Albrook Field flows the Rio Curundu, which drains a large area and carries considerable sewage from Panama City and Ancon. This stream had been previously put in a culvert from the old Gaillard Highway to the east boundary of Albrook Field, but through the field it flowed in an open ditch, a distinct flying hazard. It was of major importance that the culvert be continued through the field. The existing culvert was extended in a double box 500 ft long and then continued in an arch section 90 sq ft in area and 1,900 ft long.

As shown in the vertical airplane photograph, Albrook Field may be considered to consist of three parts: The area lying north of the old Gaillard Highway, or the North Field, crowned at elevation 19.0; the area immediately south of this highway and extending to the Curundu Culvert, or the Center Field, crowned at elevation 22.0; and the area from the culvert to the southern boundary, known as the South Field, also crowned at elevation 19.0. Owing to the presence of a high tension line, which parallels the Panama Railroad to Balboa, the South Field cannot be used as a landing field, except for emergencies, so that the usable field extends only from the Curundu Culvert north.

In June of 1930 a contract was let for the filling in of the area north of the old Gaillard Highway and for subsurface drains under the entire field. Working from the crowns, the fields were sloped one-half of 1 per cent to the sides. The first drains were laid out 200 ft from the crown in the form of a square and the rest in successive squares at 200-ft intervals. At the corners or intersections were placed junction boxes to collect the water from the halves of the two sides of every square, and these in turn were connected to the junction boxes on the successive 200-ft drains by laterals, and so carried into a main. Various drains on the outskirts of the field were carried directly into open ditches, which were dug around 75 per cent of the field. These drain into the Maria Sala, Curundu, and Quebrada Plata rivers.



Albrook Field, a Reclaimed Swamp

Highways Relocated, Water Courses Routed Underground, and Drainage System Installed

The drains consist of double-strength vitrified clay pipe ranging in diameter from 4 to 15 in., laid with open joints. These were placed in ditches excavated to a minimum of 2 ft and a maximum of 3 ft below the surface, and made 24 in. wide for the 4 to 10-in. pipe and 30 in. wide for the 12 and 15-in. pipe. Crushed rock varying from 1 to 3 in. in size was placed in the trench around the pipe to within 6 in. of the surface. The remaining 6 in. was filled in with smaller stone ranging from $^1/_4$ to $^1/_2$ in. In view of the heavy rainfall often experienced, the highest record being 3.72 in. in one hour, the drains were designed not to carry off the rain as it fell, but to assure a fairly dry field one hour after the end of the downpour. The success of this method was in evidence in 1932, when planes were able to land and take off within an hour after the worst rain.

In four out of seven of the drains and junction boxes opened after two years of service, no silt was found in either pipe or box. One 6-in. drain was one-third full of silt fairly well packed, but the other two, both 8-in. drains, had about 1 in. of loose silt in the bottom. The rock fill over the pipes was in excellent condition and showed that little or no silt had entered through the joints to clog up the system. An idea of the quantity of drain pipe used may be had from the following figures:

Diameter	Length
4 in.	22,000 lin ft
6 in.	27,700 lin ft
8 in.	22,900 lin ft
10 in.	3,450 lin ft
12 in.	1,350 lin ft
15 in.	2,600 lin ft
Total	80,000 lin ft, or over 15 miles

After three years the hydraulic fill showed an almost uniform settlement of about 0.6 ft over the entire area, whereas the dry fill showed no perceptible settlement.

To take care of the drainage of the future flying line,

Officers' Houses

A Barrack for 200 Men

TYPES OF PERMANENT QUARTERS BUILT

a grated box culvert 18 in. wide, varying in depth from 18 to 36 in. and having a length of 2,500 ft, was extended the full length of the proposed five hangars.

TWO BRIDGES BUILT TO RELOCATE GAILLARD HIGHWAY

To complete the field and make it a unit required the elimination of the Gaillard Highway, which carries considerable traffic. The question was whether to route it through Balboa or Ancon. The latter routing having been decided on, the road was relocated around the field on the right of way of the Panama Railroad. The highway is of reinforced concrete 18 ft wide, widened and super-elevated on curves for a speed of 25 miles an hour. The thickness of the concrete is 7 in. at the crown, increased to 9 in. at the sides. Transverse joints are spaced at intervals of 61 ft 6 in., and a dummy joint is carried down the center. Shoulders were made 5 ft wide, and compaction was accomplished by loading a 5-ton dump truck with rock and using a rear wheel as a roller.

To cross the Rio Maria Sala and the Rio Curundu two reinforced concrete bridges with spans of 10 and 12 ft, respectively, were built. Their width was made sufficient (33 ft) to allow for the widening of the highway. Piles for the Rio Curundu bridge were driven to refusal at elevation −16.0, and those for the Rio Maria Sala bridge, to elevation −40.0.

Considerable difficulty was experienced in filling in the stretch of road between the two bridges, which was in soft ground. The original cross sections called for 5,000 cu yd, more or less, of fill, but actually some 25,000 cu yd were used. Since the stretch still showed settlement when the question of pavement was being considered, it was decided to macadamize it until settlement had stopped. One year later, after considerable additional dirt had been thrown in by the Canal authorities, this strip was concreted and to date has shown little settlement. The total length of the relocated road is $2^{1}/_{5}$ miles; that is, it is 900 ft longer than the old route.

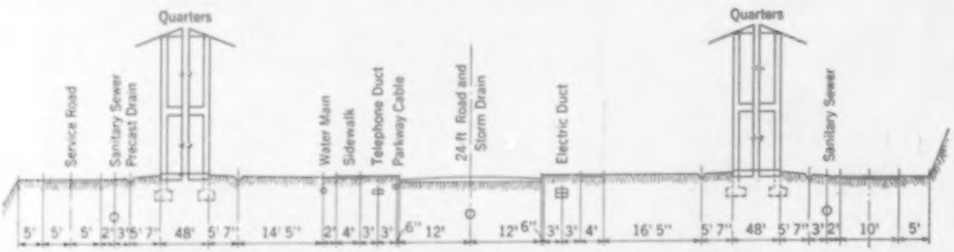


FIG. 1. SECTION THROUGH A POST STREET, SHOWING UTILITIES

The quarters of the officers and enlisted personnel are in the hills to the northwest of the flying field. Barracks to house a total of 620 men have been built facing the old Gaillard Highway. They constitute four buildings, grouped in the shape of a crescent and placed from 75 to 125 ft from the road. Space has been left in the line for one additional 110-man barrack, to be built when additional funds are available.

The quarters area was divided into two parts, for the non-commissioned and commissioned officers. For the sake of economy and to obtain the greatest number of quarters for the money available, excavation was kept at a minimum. This called for street grades up to 8 per cent and some rather bad horizontal curves, but since the post has been landscaped and cleaned up the curves seem justified. The post roads are 24 ft wide with a 2-in. crown. In thickness they vary from 6 in. at the crown to 8 in. at the sides. On both sides there are sidewalks 4 ft wide and 6 ft from the curbs.

SANITARY AND STORM SEWERS PROVIDED

Storm sewers large enough to take care of the heaviest showers are located in the middle of the streets. Sanitary sewers are carried back of the quarters and run into mains, where they connect with the barracks and then empty into the Rio Maria Sala. In Fig. 1 is shown the location of the utilities. The tide backs up in the Rio Maria Sala as far as the barracks, so that fecal matter is deposited on the banks at low tide with the resulting objectionable odors. Therefore in the very near future the Sala will have to be enclosed in a culvert connecting with the Rio Curundu culvert and thence with the culvert under the Panama Railroad, which flows into the Canal itself. Electricity for light and power is carried in a duct system leading from the trans-Isthmian duct line of the Panama Canal adjacent to the field. The distribution panels at present are housed in a temporary shanty but will be placed permanently in the Guard House on its completion.

During the first few months of operation of the post it was shown that surface drains were to be considered not a luxury but an urgent necessity if any of the landscaping was to be retained. Precast drains of a type developed and successfully used by The Panama Canal were installed. They consist of half cylinders of concrete 3 ft long and 13 in. in diameter, laid on the surface around all buildings to catch roof run-off and drip, and at the foot of slopes and in the bottom of ditches to prevent excessive erosion. Drains were carried into open ditches, streets, and catch basins.

Cutting a Ditch in Hydraulic Fill

Vitrified Clay Pipe and Rock Backfill

A Typical Junction Box

PERMANENT DRAINS INSTALLED

Service roads of water-bound macadam, 10 ft wide and 8 in. thick, conforming to the best practice in the United States, were constructed in the rear of the quarters of non-commissioned and commissioned officers. These roads serve the dual purpose of garage entrances and service roads for garbage and post service trucks.

WATER SUPPLY FACILITIES

To obtain and maintain an adequate water supply, the Army and the Panama Canal authorities worked together to establish a system that would be of mutual benefit. Three tanks of 1,500,000-gal capacity each, 90 ft in diameter and 33 ft high, were decided on. To date funds have permitted the construction of two of them. Engineer's Hill, northwest of Albrook Field, on the Army Post of Corozal, was the site selected. The hill was cut off from an elevation of 350 ft to that of 293 ft to obtain an area large enough for the three tanks originally contemplated.

Water is pumped from the Balboa Pumping Station, a distance of about two miles, and lifted from an elevation of about 15 ft to the tanks. Thence it is distributed to and through Albrook Field in a 16-in. cast-iron main, which continues to Panama City. The Albrook Field tap is a 10-in. line, which is carried into a control house where the pressure is reduced from 125 to 50 lb per sq in. From the control house the 10-in. main branches to the quarters area and to the barracks and technical area. All mains from 6 to 10 in. in diameter are of iron centrifugally cast. House connections, which vary in diameter from $1^1/_2$ to $2^1/_2$ in., are of galvanized wrought-iron pipe. The water connections were laid parallel to the front of the houses near the sidewalks.

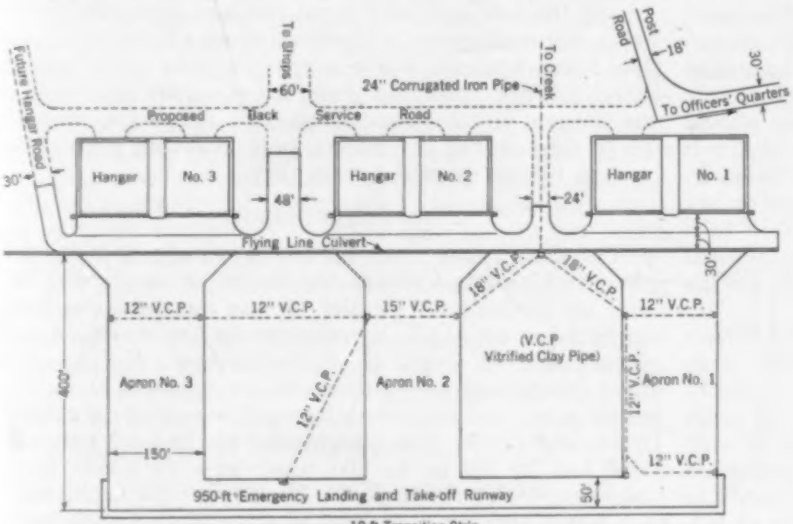


FIG. 2. LAYOUT OF HANGARS AND APRONS

The telephone ducts, which were installed by the contractor for the whole area, were placed in the strip between the walk and the curb on the left side of the road, and the electric ducts were placed in a similar position on the right side. The parkway cable for street lighting lies along the curb about 18 in. below the surface.

TERMITE-PROOF BUILDINGS

When the design of the buildings was considered, it was decided to resort to concrete structures if for no other reason than to keep out the termite. This insect, erroneously called the white ant, is the scourge of timber construction in the tropics, and everything possible is done to keep it out. When lumber is used in constructing houses, the common practice is to place the posts on concrete foundations rising about 6 in. above the surrounding area. The bottoms of the posts are placed in channels filled with creosote.

There are several objections to this type of insulation. First, it needs constant maintenance, which is often neglected when dependence must be placed on a low class of labor; second, rain beats in on these channels and washes out the creosote; and third, dirt and leaves soon clog them up unless they are properly maintained, offering to termites an opportunity to build their covered

passageways. A more recent practice of The Panama Canal has been to build concrete foundations and columns and then insulate them from the timber parts of the

RIO MARIA SALA CONDUCTED UNDERGROUND
In a Reinforced Concrete Box Culvert 9 by 10 Ft

structure by 16-oz copper plates turned down over the columns to an angle of 45 deg to present a formidable barrier to the termites. In this type of construction the stairs are built separately from the house, and a gap of about $1^1/_2$ in. is left between the stair landing and the house. The quarters at Albrook Field were built of concrete with tile roofs. For trim and roof frames, redwood and termite-proof, chemically impregnated wood were used.

Commissioned officers' quarters were built in two sizes, one for company officers and a larger size for field officers. The houses are designed for one family each and have large rooms arranged for tropical conditions. The bachelor officers are taken care of in one large apartment house, consisting of 16 sets of quarters of two rooms and a bath each. A dining room, library, and billiard room are also provided. Servants' rooms and garages occupy the first floor. The quarters of non-commissioned officers are two-story, two-apartment units.

HANGARS AND APRONS PROVIDED

In 1931 the contract for the foundations of hangars Nos. 2 and 3 and the Air Corps Warehouse and shops was let. Since the back of hangar No. 1, the third hangar to be built, would reach to the middle of the Rio Quebrada Plata, and because it was desired that this hangar should be on a line with the others, the installation of a 250-ft culvert, 7 by 7 ft in section, was a necessity. The hangars, of a type similar to those erected in the States, are 120 by 240 ft. The Air Corps shops are able to take care of all repairs required on the field. Their equipment consists of planers, lathes, drills, air compressors, milling machines, and smaller equipment.

Since Albrook Field was designed as a base for pursuit ships, it was necessary in laying out the warming-up aprons to provide for 25 or 30 ships per hangar, all warming up at approximately the same time. Pursuit ships are about 30 ft from nose to rudder and have a wing spread of about 25 ft. These dimensions vary somewhat with different types. The length of apron selected to provide available space for the average number of machines was 400 ft, and the width, 150 ft. As shown in Fig. 2, this width allows ample space for parking planes at both sides, running planes in and out, and operating a gasoline truck, all at the same time.

The concrete was designed for a gasoline truck weighing about $7^1/_2$ tons when fully loaded, it being the heaviest load expected on the aprons. Although bombers also use these aprons, they are somewhat lighter than the truck. Aprons, roads, and ramps were reinforced transversely with $^1/_4$-in. round bars 6 in. on centers and longitudinally with $^3/_8$-in. round bars from 14 to 15 in. on centers. The slabs in the aprons are 10 by 50 ft and 6 in. thick, with metal contraction joints of the standard highway type and $^1/_2$-in. premolded asphaltic joints on the shorter side. The 30-ft road is 6 in. thick at the crown and 8 in. at the sides, but the ramps have a uniform thickness of 7 in.

In building the aprons the contractor elected to grade all three at once and then pour alternate 10-ft slabs simultaneously. For instance, in Apron 1 he poured Slabs 1 and 3 the first day; 5 and 7, the second; 9 and 11, the third; and 13 and 15, the fourth. When these slabs were properly cured, the intermediate ones were poured. Concrete was cured by spraying emulsified asphalt on the fresh surface. One reason for using this material was to kill the glare of the concrete.

To secure rapid run-off from the center of the aprons it was decided to use a hyperbolic curve, according to the formula:

$$y = \frac{c}{6}\left(-5 + \sqrt{25 + \frac{384x^2}{W^2}}\right) \quad . . . . [1]$$

in which y is the difference in the surface elevation at the crown and at a distance x from the crown; W is the total width of the roadway; and c, the total height of the crown. Results have been very gratifying. Drainage was accomplished by turning up the outer 2 ft of the aprons around the edges.

After the rainy season of 1932, when planes attempted to land while the field was still too wet, the three aprons were connected with a 50-ft concrete strip, thus providing a 950-ft emergency landing and take-off strip available in rainy weather for all planes except bombers.

Around Albrook Field the country consists of low rolling hills running as high as 1,500 ft, and heavily wooded. The valleys are covered with a saw grass known locally as *cana brava*, which spreads like wildfire. When it grows its roots bunch up and form clumps. Its eradication is difficult and can only be accomplished by hand if kept at day by day. The best grass with which to sod the field and keep down the *cana brava* was found to be Bermuda grass. After two years a fairly good mat has been obtained which keeps the *cana* quite well in hand. When Bermuda grass is cut up by tail skids it roots again and continues growing. It has the strength to go through the dry season, only browning up a little, and so far has proved satisfactory.

ACKNOWLEDGMENTS

The design and plans for the hydraulic and dry fill, the Rio Curundu Culvert, the drains on the flying field, and the preliminary studies for the quarters area were carried out by M. J. Newman under the direction and supervision of Capt. B. F. Vandervort, U.S.A. The final design and construction of the quarters area, barracks area, technical area, hangars, warming-up aprons, roads and ramps, and the grassing of the field were carried out by me under the supervision of Capt. B. L. Meeden, U.S.A., with R. C. Buckley as the architect.

Contractors for the various parts of the work were: hydraulic fill, Dredging Division of The Panama Canal, John G. Claybourne, superintendent; dry fill and field drainage, Compania Constructora Nacional; quarters, barracks, and utilities, hangar and shop foundations, J. A. Jones Construction Company; reservoirs, relocated Gaillard Highway, service roads, and precast drains, Grebien and Martinz; erection of hangars and shops, J. W. Patience; and warming-up aprons, roads, and ramps, Tucker T. McClure.

Influence of Temperature on Coagulation

Water Treatment Experiments in New Jersey Show Floc Formation and Color Removal Are Retarded with Rising Temperatures

By Clarence J. Velz

Associate Member American Society of Civil Engineers

Sanitary Engineer, Federal Emergency Administration of Public Works, Newark, N.J.

Contrary to the generally accepted view that chemical activity increases with a rise in temperature, experiments in water treatment recently conducted by Mr. Velz indicate the exact opposite. His tests showed strongly that the dosage of coagulant required to produce a water color of 20 ppm decreased with a rise in temperature. Floc formed more quickly when the temperature of the raw water was low—from about 2 to 14 C. At summer temperatures larger dosages of coagulant were uniformly required to produce the same results. An analysis of such factors as the hydrogen ion concentration, the original color of the raw water, and other influences which might happen to coincide with the seasonal change in temperature bore out the strong positive correlation between temperature and amount of coagulant required to produce a standard water. Further experimentation will be needed to verify Mr. Velz's suggested explanation, based on electro-chemical activity.

Considerable research work on water purification has been conducted in recent years in an attempt to discover the fundamental factors influencing floc formation and color removal. From these contributions has come recognition of the significance of the hydrogen ion and also a better understanding of the process of color removal. In his pioneering work on the electro-chemical phases of color removal, Thorndike Saville, M. Am. Soc. C.E., and many workers since have changed the concept of the process from one of simple mechanics to one of an electro-chemical complexity. More recently, attention has been focussed on the influence of a number of ions other than those of hydrogen, but knowledge to date concerning them is rather limited. Many phenomena are explained by assuming that each water is a law unto itself and by attributing its peculiarities to inherent characteristics.

During 1932–1933 I was in charge of an experimental water purification plant, of 30,000 gal per day capacity, which was operated primarily to study the effect of mixing and conditioning in the treatment of the potable water supply, and of improving the operation of the existing 35 mgd plant. The source of supply was surface water of a drainage area in northeastern New Jersey. Although the experimental plant was operated largely to secure more economical operation rather than to study fundamental principles, certain observations were made which may be of general interest and value in water works engineering.

One of the startling facts discovered in this work was the predominant rôle played by temperature in plant operation. Contrary to the general conception, higher temperatures had a detrimental effect, for they necessitated a substantial increase in the amount of coagulant required. Three independent studies on temperature were made and are presented here.

BENCH TESTS IN WINTER AND SUMMER

The first observation of the effect of temperature was noted in connection with jar experiments using a bench mixing machine. Jar tests were made regularly to determine the minimum dosage of coagulant to be used in operating the experimental plant. A marked change in the required amount of dosage was noted with the advent of warm weather. During the winter months the quantity of alum required to produce a basin effluent color of 20 ppm appeared to be a function of the color of the raw water. From the bench test data collected during this period a curve indicating this relationship was constructed, which served as a guide for plant operation. This curve, designated "winter curve," is shown on Fig. 1. The alum requirements throughout the late fall and winter did not deviate appreciably from the amounts indicated on this graph. However, as soon as the temperature of the raw water rose to 16 C a marked change was noted, and the deviation from the winter curve became more pronounced as the temperature rose.

As measured by routine plant procedure, the variation in the chemical characteristics of the raw water was not sufficiently marked to explain the decided change in alum requirements. An analysis of the bench test data on raw water having a color from 34 to 38 ppm, for temperatures from 8 to 24 C, indicated a close relationship between temperature and amount of alum dosage, as shown in Fig. 2. From this figure it is apparent that the influence of temperature was not marked during the winter, when the temperature was between 8 and 14 C. In that range the amount of alum required varied only between 0.9 and 1.1 grains per gal. However, between 14 and 24 C, there was a decided change in the amount of alum required, which ranged from 1.1 to 2.1 grains per gal. This variation appeared to be definitely correlated with temperature.

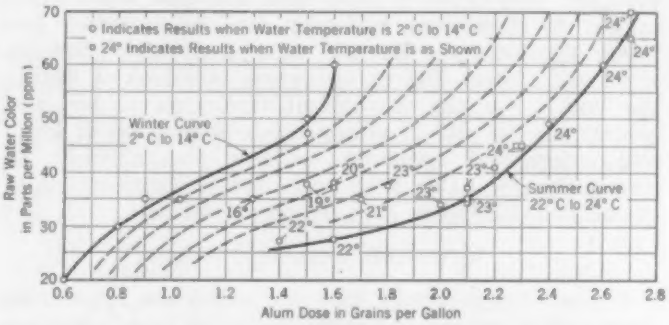


Fig. 1. Relationship Between Raw Water Color and Alum Dose Required to Produce a Color in the Basin Effluent of 20 ppm at Temperatures Shown

From Jar Tests with a Bench Mixing Machine

Results obtained in the experimental plant paralleled those secured in the jar tests and indicated the same relationship between alum requirements and temperature. The jar tests with the mixing machine followed

routine procedure. The dosage for the experimental plant was found to be approximately 0.85 times that used in the jar tests.

When the data for the summer period, May to September, are plotted on the same graph with the winter data, the temperature influence is startlingly apparent (Fig. 1). The marked difference between the winter and the summer curves cannot be attributed to the raw water color, which varied in both seasons from 30 to 70 ppm. The summer dosage is found to be from 1.6 to 2.2 times that of the winter dosage.

Temperature and raw water color were the two dominating factors which indicated the economical dosage of

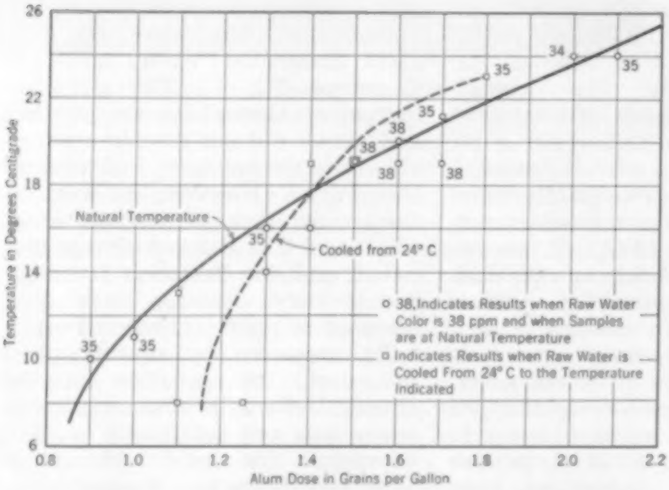


FIG. 2. RELATIONSHIP BETWEEN TEMPERATURE AND ALUM DOSE REQUIRED TO PRODUCE A COLOR IN THE BASIN EFFLUENT OF 20 PPM WHEN RAW WATER COLOR IS AS SHOWN

From Jar Tests with Bench Apparatus

coagulant. The relationship between alum dosage, water temperature, and raw water color may be represented by a series of curves lying between the winter curve and the summer curve in some such way as indicated by the dotted lines in Fig. 1. The general characteristics of this type of curve are fixed primarily by the color of the raw water, whereas the position of the curves on the alum dosage axis is a function of temperature.

The strong correlations between alum dose and temperature, and between alum dose and raw water color appear to show that hydrogen ion concentration has minor significance. The pH value had varied through the same ranges during the warm weather as during the cold. For the method of treatment under consideration—the use of alum, or a combination of alum and chlorinated copperas, without any attempt to adjust the pH value with acid—the fundamental relationships indicated in Figs. 1 and 2 hold true.

ARTIFICIAL COOLING TESTS

It may be protested that the influence apparently due to temperature, indicated in Figs. 1 and 2, was not actually the direct result of temperature but rather was caused by a variation in some unknown factor which happened to coincide with the seasonal change. A series of cooling experiments conducted on the raw water in the summer substantiated the relationships derived from the bench test data and strongly indicated that temperature and not some coincident factor was responsible for the reactions observed.

These experiments commenced with the warm summer water, which was gradually cooled to winter temperatures so as to determine whether the relationship previously determined held true in this case. The bench mixing apparatus was used in these tests, and the alum requirements were measured as usual. One series was run on the raw water without altering the temperature, and the other on the raw water cooled to a definite temperature and held practically constant in an ice bath. In every test a marked superiority was shown by the cooled series when compared with the uncooled series as regards color removal, speed of floc formation, and quantity and quality of floc. The original water samples were at 24 C. Although the curve of alum dosage for the cooled samples, the dotted curve in Fig. 2, does not coincide exactly with the solid curve for the samples tested at natural temperatures, the amount of alum required definitely grew less as the temperature went down. This indicated with reasonable certainty that temperature actually was the cause of the variation in the amount of alum required.

It must be remembered that the influence of temperature is apparent only when the minimum economical alum dosage is used. When the dose is more than the minimum required to remove the color economically and practically, the effect of temperature is greatly reduced. In much research work therefore, the temperature factor has been unrecognized, because in general the dosages of coagulant have been much larger than those required for normal economical plant operation.

MATHEMATICAL COEFFICIENTS OF RELATIONSHIP

If the influence of temperature is as strong as indicated by the two types of evidence discussed up to this point it is logical to assume that this fact will be reflected in the data of the 35-mgd main plant and can thus be measured quantitatively by standard statistical methods. An analysis of means, standard deviations, and coefficients of correlation, both multiple and partial, substantiates the experimental observations and places a high significance on temperature influence. An elementary discussion of correlation is given by W. P. and E. M. Elderton in Chap. 5 of their *Primer of Statistics* (A. and C. Black, London). A good treatment of the technique of computation may be found in Chap. 9 of *Statistical Methods Applied to Education*, by H. O. Rugg (Houghton Mifflin Company, Boston). The mathematical theory of correlation, essential to an understanding of the method, is well presented in Chap. 2 of *Forecasting the Yield and the Price of Cotton*, by H. L. Moore (The Macmillan Company, New York); and in Chaps. 9 and 10 of *An Introduction to the Theory of Statistics*, sixth edition, by G. U. Yule (C. Griffin and Company, Ltd., London). The logical basis of correlation is best discussed by Karl Pearson in Chaps. 4 and 5 of his *Grammar of Science* (A. and C. Black, London). An excellent example of the application of the technique of partial and multiple correlation is contained in a monograph by Frank A. Ross, *The Method of Partial and Multiple Correlation Applied to School Attendance* (Census Monograph 5, Bureau of Census, Washington, D.C., 1924). A graphical solution of a correlation table is given in TRANSACTIONS, Vol. 94, 1930, pages 936–960.

The data for the main plant for the period from June 1932 to June 1933, shown in Table I, were selected as comparable with those from the operation of the experimental plant. A five-day average practically coincided with the changes in plant operation. The major factors influencing alum dosage, X_1, were taken to be the

following: color of raw water, X_2; temperature of raw water, X_3; and pH value of raw water, X_4. The mean alum dose for the period is computed to be 1.15 grains

TABLE I. DATA PERTAINING TO THE OPERATION OF THE 35-MGD MAIN PLANT

Five-Day Averages, June 1932 to June 1933

Date	Raw Water			Alum Dose
	Color, ppm	pH	Temperature F	Grains per Gal
June, 1932	30	7.3	68	1.30
	28	7.3	68	1.31
	27	7.5	68	1.27
	35	7.5	71	1.28
	32	7.4	69	1.27
	28	7.5	72	1.27
July	36	7.5	73	1.34
	30	7.6	72	1.40
	38	7.8	74	1.42
	38	7.6	74	1.40
	38	7.6	75	1.44
	36	7.6	76	1.43
August	29	7.6	76	1.42
	30	7.7	76	1.40
	32	7.5	76	1.43
	37	7.5	76	1.42
	38	7.7	76	1.42
	36	7.5	76	1.50
September	30	7.6	78	1.39
	33	7.6	73	1.41
	34	7.8	67	1.42
	34	7.7	65	1.42
	35	7.6	68	1.42
	31	7.7	64	1.95
October	36	7.4	55	1.00
November	42	7.4	50	0.95
	42	7.4	50	0.96
	47	7.3	47	1.07
	42	7.2	44	1.22
	52	7.2	40	1.43
December	48	7.1	40	1.44
	42	6.9	40	1.26
	37	6.9	38	1.26
	36	6.9	35	1.17
	34	7.0	36	1.15
	33	7.1	37	1.20
January, 1933	32	7.1	36	1.18
	28	7.1	37	.00
	25	7.2	37	1.00
	25	7.2	37	0.78
	24	7.3	39	0.82
	25	7.4	38	0.78
February	24	7.6	38	0.77
	23	7.6	36	0.73
	25	7.6	33	0.67
	24	7.6	35	0.67
	23	7.4	37	0.67
	30	7.1	36	0.67
March	25	7.0	38	0.70
	29	7.1	38	0.98
	32	7.3	38	0.88
	31	7.4	39	1.00
	32	7.3	39	0.99
	32	7.2	40	0.99
April	29	7.2	44	0.78
	29	7.3	48	0.74
	34	7.4	50	0.92
	38	7.1	52	1.04
	41	7.1	52	1.10
	40	7.2	53	1.13
May	36	7.3	57	1.15
	35	7.3	59	1.26
	37	7.2	58	1.17
	38	7.4	61	1.13
	40	7.4	64	1.13
	37	7.3	68	1.28

per gal and the standard deviation, 0.25. The standard deviation, σ, is the square root of the arithmetic mean of the squares of all deviations, deviations being measured from the arithmetic mean of the observations. The means and standard deviations for the other factors are as follows:

Alum dose, X_1 = 1.15; standard deviation, σ_{X_1} = 0.25
Color of raw water, X_2 = 34; standard deviation, σ_{X_2} = 6.15
Temperature, X_3 = 54 F; standard deviation, σ_{X_3} = 15.5
pH value, X_4 = 7.35; standard deviation, σ_{X_4} = 0.22

In general the opinion is that raw water color and its pH value are the factors which determine alum dosage,

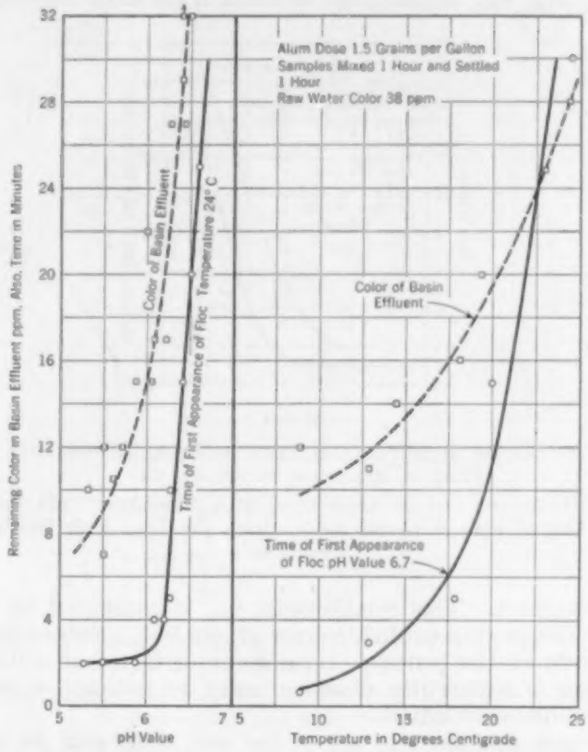


FIG. 3. COLOR REMOVAL AND TIME OF FIRST APPEARANCE OF FLOC IN RELATION TO BOTH TEMPERATURE AND pH VALUE

1.5 Grains per Gal of Alum Used as a Coagulant; pH Value Adjusted with Sulfuric Acid; from Jar Tests with Bench Apparatus; Temperature Adjusted by Cooling in Icebox

and therefore the first coefficient of relationship (r) computed was between alum dosage and color of raw water. The relationship is expressed by the general equation,

$$r_{X_1 \cdot X_2} = \frac{\Sigma x_1 \cdot x_2}{N\sigma_{X_1} \cdot \sigma_{X_2}} \qquad \ldots\ldots [1]$$

which is a derivation of Galton's law of simple relationship, where x_1 and x_2 are deviations of the individual values from the means of the X_1 and X_2 series, respectively. In it r may vary from -1 to $+1$. The relationship is weak when near zero and strong when near unity. Unity is taken as a perfect relationship. The coefficient of relationship, r_{12}, between alum dose, X_1, and color of raw water, X_2, is computed to be $+0.421$, with a probable error of ± 0.10. This relationship is not strong, since four times the probable error practically equals the coefficient. This is somewhat surprising as it indicates that the color of the raw water is not a strong factor in determining what the alum dosage shall be throughout the year. Likewise, the relationship, r_{14}, between alum dose, X_1, and the pH value of the raw water, X_4, is computed to be $+0.292$, with a probable error of ± 0.076. Here again, four times the error practically equals the coefficient, and hence the variation in the pH value of the raw water

has but little significance. The difference between the squares of the coefficients indicates that the color influence is about twice as strong as the influence of the pH value but neither of these two coefficients is significant. From a statistical standpoint they are relegated to positions of minor importance. The primary determinants must therefore be other factors.

That temperature is the dominant factor is borne out by the high coefficient of relationship between the alum dose and the temperature derived from data from the

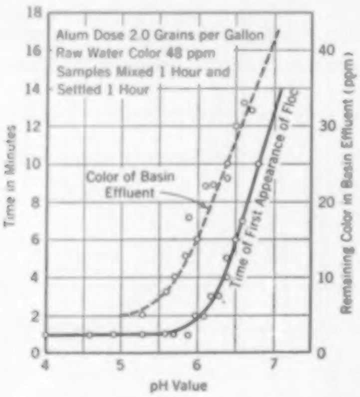


FIG. 4. COLOR REMOVAL AND TIME OF FIRST APPEARANCE OF FLOC IN RELATION TO PH VALUE

2.0 Grains per Gal of Alum Used as a Coagulant; pH Value Adjusted with Sulphuric Acid; from Jar Tests with Bench Apparatus

main plant. This relationship, r_{13}, is computed to be $+0.750$, with a probable error of ± 0.054. Subtracting four times the probable error leaves a factor of $+0.53$, which is sufficiently close to unity to indicate a considerable correlation.

These relationships are of the zero order and do not take into account the interrelationship of the several factors. Hence alum dose, raw water color, and temperature were selected as three variables, and computations for the partial and the multiple relationships were made. The partial coefficient, $r_{12 \cdot 3}$, which is the relationship between alum dose, 1, and raw water color, 2, when temperature, 3, is held constant, is computed to be $+0.395$. This is less than the zero order, r_{12}, of 0.421, showing that temperature is involved in r_{12}. Similarly, the relationship, $r_{13 \cdot 2}$, between alum dose and temperature, with raw water color held constant, is computed to be $+0.742$. This is but slightly less than the value of r_{13} previously computed to be $+0.750$. Also the relationship, $r_{23 \cdot 1}$, between raw water color and temperature, with alum dosage held constant, is computed to be -0.155. This brings out markedly the strength of the relationship between temperature and alum dosage.

DEMONSTRATING THE RELATIVE IMPORTANCE OF TEMPERATURE AND RAW WATER COLOR IN DETERMINING ALUM DOSE

Now, by combining the relationship between alum dose and raw water color plus temperature, a multiple coefficient, $R_{1(23)}$, of $+0.793$ is computed. The inclusion of the raw water color in the multiple coefficient increased the relationship between alum dose and temperature only from 0.750 to 0.793, whereas inclusion of the temperature factor in the relationship between alum dose and raw water color increased the coefficient from 0.421 to 0.793. This shows to some extent the relative importance of temperature and raw water color. Since a zero order coefficient of 0.421 changes the temperature coefficient only from 0.750 to 0.793, it is obvious that to include in the multiple coefficient a zero order of 0.292 for the pH value of the raw water will raise the factor 0.793 but slightly. Therefore, the laborious mathematical technique involving four variables was not undertaken.

These mathematical coefficients of relationship are consistent with the results of observations made at the experimental plant. The low coefficient of correlation between alum dose and pH value is in substantial agreement with the experimental results. Careful observation during changes in the pH value of the raw water soon led to the conclusion that the variation in the concentration of the hydrogen ion of the raw water had but little effect on operation or results. This was particularly true during cold weather. The coefficient of correlation of 0.421 between alum dose and raw water color appears to be too weak to be consistent with Fig. 1, when taking into consideration the entire range of raw water color from minimum to maximum. However, an analysis from the standpoint of the mean color and the standard deviation from the mean shows no disagreement. The mean color for the year was 34 ppm, with a standard deviation of 6 ppm. From Fig. 1 it will be apparent that a variation in the color of the raw water of plus or minus a standard deviation from the mean only alters the alum dose from 15 to 20 per cent.

The coefficient of 0.75 for the relationship between alum dose and temperature certainly substantiates the conclusions derived from the experimental data, and the relative strength of the influence is consistent with the tests. A standard deviation in temperature from the mean produced a 60 per cent change in alum requirements. The data from the operation of the experimental plant, those from the cooling experiments, and those from the main plant all concur in placing major importance on temperature. This can be said to be true only for this particular water and for this particular method of treatment.

RELATIONSHIP BETWEEN TEMPERATURE AND PH VALUE DISCUSSED

By an adjustment of the pH value with acid to the optimum point, the detrimental effect of high temperature can be practically eliminated. It appears from controlled tests, using the bench mixing machine, that the optimum concentration of hydrogen ions is related to temperature. At summer temperatures the optimum is between narrow limits, in the vicinity of a pH value of 5.4, whereas in the winter the optimum has a considerable range. Excellent results can be obtained at a pH value of 6.7, when the temperature is between 8 and 14 C, but when the temperature is between 20 and 25 C it is necessary to adjust the pH value to a point below 5.8 to get comparable results with the same quantity of coagulant.

When alum is used as a coagulant, rapid floc formation, in general, indicates good color removal and good agglomeration. Rapid floc formation can be obtained either by adjusting the pH value to a point below 5.8 or by controlling the temperature to a point below 16 C. In other words, if the temperature is below 16 C, an adjustment of the pH value is not necessary, but if the temperature is much above 16 C, it is necessary to increase the hydrogen ion concentration in order to get the same results. A typical indication of the relationships involved is presented in Fig. 3. From this graph it is seen that the time of floc formation and color re-

moval tend to parallel each other. At summer temperatures and at a pH value above 6, floc forms slowly and color removal is poor. The best results are obtained at a pH value of 5.4, when floc forms within two minutes by jar tests and mixing machine, and color removal is the maximum possible with the dosage used. A comparison of Fig. 3 with Fig. 4 shows that the smaller the quantity of coagulant used, the more steep are the color and time curves, and therefore the narrower the zone of pH values that will produce economical coagulation. The specific observations presented in Figs. 3 and 4 are typical of the general relationships which controlled reactions throughout the entire period of the operation of the experimental plant and appear to be in agreement with the classical research of Clark, Theriault, Miller, and more recently, of Bartow, Black, and Sansbury.

These observations hold for summer conditions. During the cold weather quite another condition prevails which is caused by temperature. Reference to Fig. 3 will show how important temperature is in floc formation and color removal. When the temperature is at 24 C and the pH value is 6.7, floc formation is slow and the amount of color removed small, but as the temperature is reduced, either by the natural seasonal processes, or by cooling artificially, the speed of floc formation increases and color removal is more complete. As the temperature approaches the freezing point, floc formation is almost instantaneous, agglomeration is excellent, and color removal approaches its maximum.

RESULTS OBTAINED WITH CHLORINATED COPPERAS AS A COAGULANT

The effect of temperature observed when alum was used as a coagulant was also apparent to the same extent when a combination of 70 per cent alum and 30 per cent chlorinated copperas was used. When chlorinated copperas alone was used the temperature influence was also apparent. However, only a few tests were made and therefore no definite conclusions are presented. For the relationships of the pH value of the treated water to the time for the first appearance of floc and to the units of color removed, a more extended series of tests was made, using chlorinated copperas alone. The results are indicated in Fig. 5.

Floc formed rapidly on the acid side between pH values of 3.8 and 4.8; also on the alkaline side between pH values of 8.0 and 10.0. The quality of the floc and the agglomeration were excellent on the acid side, but on the alkaline side the floc never agglomerated into large particles, even with prolonged mixing. Likewise, the water was clear when on the acid side and cloudy and colloidal when on the alkaline side. These observations are similar to those made by Bartow, Black, and Sansbury. The color removal curve, when chlorinated copperas was used as a coagulant does not parallel the curve showing time for first appearance of floc to the same extent as it does when alum was used.

On the acid side, between pH values of 3.8 and 4.8, the speed of floc formation could be used as an index for effective color removal, but on the alkaline side, although floc formed rapidly, there was no removal of color whatever. In the zone between pH values of 4.8 and 8.0 floc formed slowly; the water was highly colloidal; and color was imparted to the water instead of being removed. Chlorinated copperas alone as a coagulant was not economically practical, as it necessitated large additions of acid in order to keep the pH value of the water within the zone where color was effectively removed. However, a combination of chlorinated copperas and alum proved to be far superior to either coagulant alone.

CONCLUSIONS

In these investigations no attempt was made to control the chemical characteristics of the water, but rather it was the purpose to determine a practical treatment of the natural surface-water supply. Variation in the chemical composition, as measured in routine water

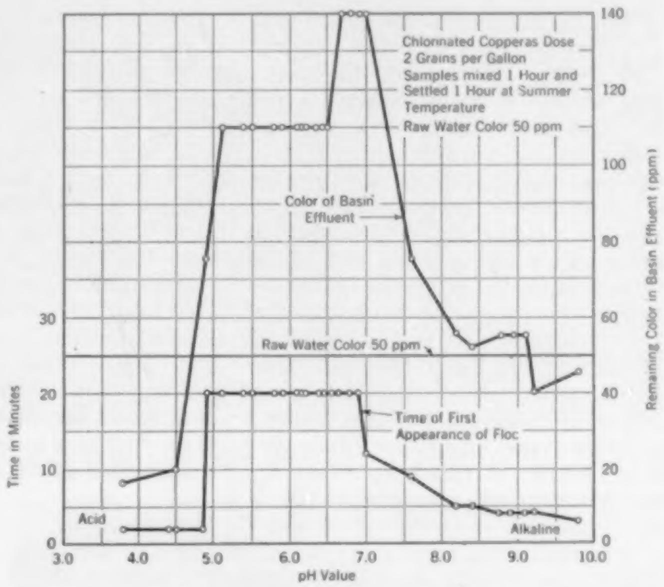


FIG. 5. COLOR REMOVAL AND TIME OF FIRST APPEARANCE OF FLOC IN RELATION TO pH VALUE

2.0 Grains per Gal of Chlorinated Copperas Used as a Coagulant; From Jar Tests with Bench Apparatus

works practice, had but slight if any effect on the process of floc formation and color removal. A statistical analysis of the chemical data from the main plant indicated the same conclusion. Preliminary experiments, varying the sulfate ion, the calcium ion, and the chloride ion, did not give results which differed more than the experimental error involved in bench test methods.

The temperature phenomenon observed presents another indication of the complexity of the process of floc formation and color removal. This relationship is undoubtedly governed by electro-chemical rather than chemical laws. The three independent measurements of the effect of temperature all give a positive correlation—that is, increase in temperature requires an increase in the amount of coagulant and decrease in temperature permits a decrease in the amount of coagulant. This positive correlation is contrary to the usual conception of the influence of temperature on a chemical reaction but is in complete harmony with the principles of electro-chemistry. My theory is that, when minimum economical dosages are used, ionization, hydrolyzation, ion absorption, and the dielectric constant of water are changed sufficiently by the change in temperature to explain the results observed. The only evidence offered in support of this theory is that the temperature influences fit logically into an electro-chemical concept. Electro-chemical experimentation was beyond the scope of my work in these experiments and therefore these observations are presented with the hope that they may be investigated by others using electro-chemical methods.

Adjustable-Blade Turbine Installation

Government Installs New Hydro-Electric Unit in Its Plant at Sault Ste. Marie, Mich.

By E. L. Chandler

Member American Society of Civil Engineers
Engineer, Price Brothers Company, Dayton, Ohio

In the vertical unit recently added to the Government's hydro-electric plant at the falls in the St. Mary's River, an electrically operated mechanism adjusts the tilt of the blades of the 3,000-hp, propeller-type turbine to give maximum efficiency under varying conditions of head and gate opening. The adjustment is not made automatically but is controlled by the plant operator from the main switchboard. In the design of this unit, special attention was given to the concrete scroll case, draft tube, and hydraucone. The high test efficiency of the unit reflected the careful design and execution of the work. This is the first unit to be completed in the reconstruction of the existing plant.

Some features of the design and construction of the hydro-electric unit placed in operation in 1932 on the St. Mary's River at Sault Ste. Marie, Mich., are of interest as marking advances in both equipment and plant details for that type of power development. The new unit is an addition to the power plant adjoining the north wall of the navigation locks at the rapids in the river. This plant is owned by the U. S. Government and is operated by the Edison Sault Electric Company under lease. As shown in Fig. 1, the unit consists of a 114-in., propeller-type, adjustable-blade turbine, which is rated at 3,000 hp under a 20-ft effective head at $128\frac{1}{2}$ rpm, and direct-connected to a 2,500-kva generator.

Although the normal daily range of operating head acting on the turbine is small, it is possible to bring about a considerable variation of head by manipulating a system of control gates across the river upstream from the plant. By this means it was possible to check the performance of the unit over a comparatively large range of conditions. Such tests of actual operation showed that the power output was very considerably in excess of the guaranteed rating, and that it reached as high as 113 per cent of the rated, or guaranteed, turbine capacity throughout varying conditions of operation.

Of particular interest from a hydraulic standpoint is the fact that exceptionally smooth operation was obtained under varying head through the complete range of runner-blade tilt. A dime balanced on edge on the generator frame remained in that position under all conditions of operation. Some results of the tests are given in the diagrams of Fig. 2, which are presented by courtesy of the U. S. Engineer Office at Sault Ste. Marie.

In the design of the new unit, particular study was given to the problem of obtaining smooth, unobstructed flow in the wheel and draft pits. The results obtained in operation appear to justify the care taken in construction. A vertical section showing the essential features of the setting is given in Fig. 1. The turbine is mounted in a concrete scroll case and discharges through a concrete draft tube, the upper part of which is lined with steel, and then through a reinforced concrete hydraucone. The draft pit is entirely free from obstructions to flow outside the limits of the hydraucone, with the exception of two piers on the center line of the unit, one extending from the hydraucone to the upstream wall of the pit and the other from the hydraucone to the downstream limit of the power house structure. There is no support other than these two piers under the lip of the hydraucone, although its maximum diameter is 32 ft. As indicated in Fig. 1, the noses of the piers are 10 ft from the center line of the unit. The shape of the hydraucone and draft chamber may be understood from the illustrations.

Suspension of the hydraucone from the roof of the draft pit necessitated a heavily reinforced structure. The design provided for very heavy beam reinforcement in parts of this roof, especially across the downstream part and around the steel-lined section of the draft tube. In the latter location, the principal reinforcing steel extends in a general up- and downstream direction and is carried past the tube on each side and into the two piers previously mentioned. These tend to serve as end supports for the peripheral beams thus embodied in the roof of the chamber.

Since the weight of the hydraucone, that of the water in the wheel pit, and that of the machinery and masonry above, all come on the draft-pit roof, both center-line piers carry considerable load. However, the reinforcing

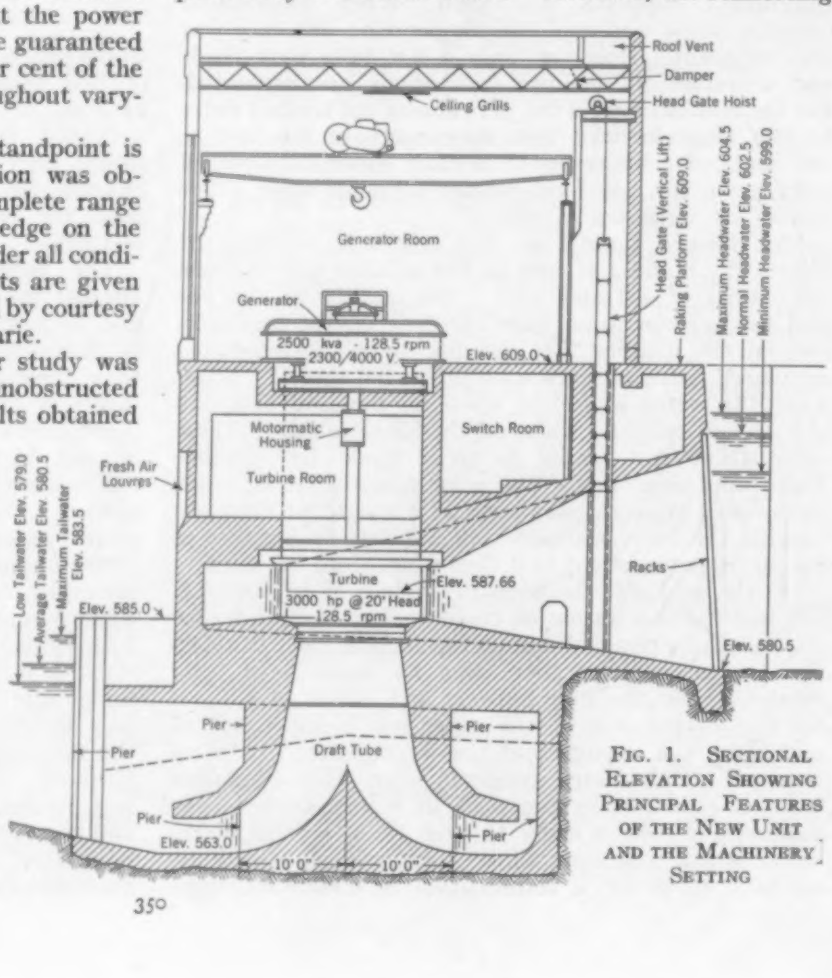
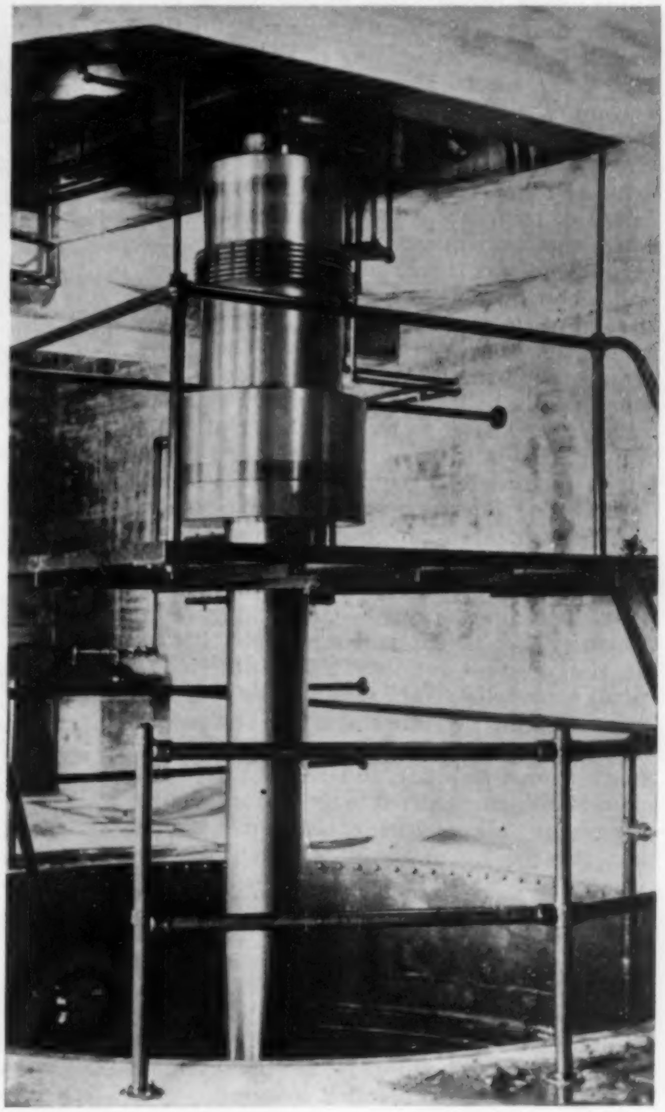


Fig. 1. Sectional Elevation Showing Principal Features of the New Unit and the Machinery Setting

THE DRAFT PIT, LOOKING UPSTREAM ALONG TAILRACE, SHOWING HYDRAUCONE AND DOWNSTREAM SUPPORTING PIER

PART OF HYDRAUCONE AND UPSTREAM NOSE OF DOWNSTREAM SUPPORTING PIER

TURBINE RUNNER AND SHAFT AS RECEIVED AT POWER HOUSE
Motormatic Equipment Under Cover at Left



"MOTORMATIC" EQUIPMENT, WHICH ADJUSTS PITCH OF TURBINE BLADES FOR BEST EFFICIENCY, UNDER CONTROL OF THE STATION OPERATOR © Allis Chalmers

NEW POWER UNIT AT SAULT STE. MARIE PLANT

steel in the chamber roof was placed so as to reduce the loads carried by these piers as much as possible. The foundation material being a hard sandstone, it was possible to design the piers with the satisfactorily small dimensions shown and obtain a very efficient chamber.

Although the adjustable-blade type of runner is not new, the method of tilt control employed in this unit is a recent development and is of interest from the point of view of plant operation. The design has been given the name of "Motormatic" by the manufacturer. There is a general resemblance to the adjustable type covered by the Kaplan patent. In the Kaplan design, however, the blade adjustment is synchronized automatically with the movement of the turbine guide vanes, transmission of movement being accomplished by oil pressure. In the unit here described, tilting of the blades is effected by electrical operation and may be accomplished either in correlation with the movement of the guide vanes or independently, through manual push-button control by the station operator at the main switchboard of the plant. In this installation the latter method appeared to be the most desirable and was therefore adopted.

The operating mechanism consists of a system of electrically operated gears housed at the top of the turbine shaft. The shaft, as in other installations of tilt-blade units, consists of a hollow member within which is fitted a solid shaft to transmit the motion of adjustment from the motor-operated gears at its upper end to the runner blades at its lower end. The essential difference between the method of tilt adjustment employed in this unit and that in other designs is that here the runner tilt is regulated by a turning movement of the inner shaft, through self-locking gears, which hold the runner blades at the position set, whereas in others the tilt is accomplished through a push-and-pull movement of the inner shaft. The motor and gears, together with their housing, are built into the shaft and revolve with it, power being transmitted to the motor through slip rings. The position of the runner blades is registered by means of a potentiometer inside the housing with leads operating back through the slip rings to a control panel in the main switchboard of the plant. A photograph of the turbine runner, taken after it arrived on the job, shows the shaft and runner mechanism as it was assembled for shipment. In this view the "Motormatic" feature is at the extreme left, hidden by a protective covering.

This type of mechanism for the operation and control of tilt blades offers many advantages for this particular installation. Several other units are in operation with the new unit, and conditions of head and load are fairly constant. Among the advantages are control of tilt to meet changes of both head and load so as to produce the power at or near the maximum efficiency of the unit at best head and gate opening. The loads range from 1,500 to 3,000 hp at a 20-ft head and corresponding power outputs for heads ranging from 17 to 22 ft. A reduction of wear on the blade bearings and other expensive parts of the turbine is effected by making no tilt adjustment for minor variations in load, over which the change in efficiency is negligible.

Careful determinations of the efficiency of the new turbine were made difficult by entrance conditions, since the deep channel of the forebay approaches the piers at an angle. However, at a head of from 17 to 20 ft the tests showed better efficiency than did either the model tests or the same sized unit installed in other plants when operating at or near the same heads.

The plant was designed by the firm of Holland, Ackerman and Holland, consulting engineers of Ann Arbor, Mich., and built by the Price Brothers Company, general contractors of Dayton, Ohio. The R. E. Townsend Corporation, of Ann Arbor, was associate contractor.

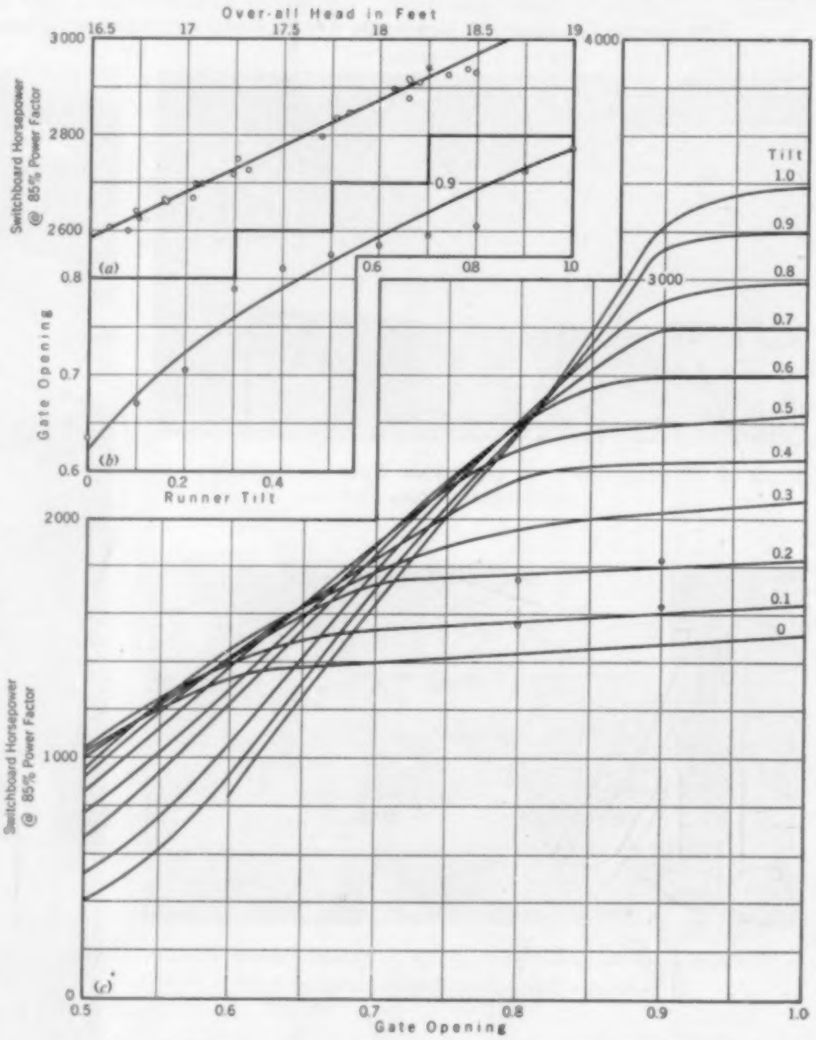


FIG. 2. TESTS OF 114-IN. PROPELLER-TYPE TURBINE AND 2,500-KVA GENERATOR ON SAME VERTICAL SHAFT, JULY 1932

(a) Head-Horsepower Curve for Full Gate and Full Tilt of Runner Blades; (b) Relation Between Gate Opening and Tilt of Blades for Best Efficiency; and (c) Relation Between Horsepower, Runner Tilt, and Gate Opening at a 20-Ft Head

Design Problems on New York Subways

UNDERPINNING THE RETAINING WALLS AT THE 167TH STREET HIGHWAY UNDER CROSSING

Complications in Upper Manhattan and the Bronx on the Now Completed 55-Mile Independent City-Owned System

By ALFRED BRAHDY

MEMBER, AMERICAN SOCIETY OF CIVIL ENGINEERS
DESIGNING ENGINEER, BOARD OF TRANSPORTATION, NEW YORK, N.Y.

MANY of the problems encountered in the building of a subway system in the city of New York are connected with the adjustment of the structure to existing improvements both above and below the ground. In general the subways are built within the street lines in open cut so that their roofs are close to the surface. In the upper part of Manhattan, however, high ground and steep slopes required tunneling methods for a part of the work, and the crossing of the Harlem River into the Borough of the Bronx necessitated a location below the river level. In this article Mr. Brahdy describes some of the problems that arose on the work under St. Nicholas Avenue and the Grand Concourse and explains how they were met.

WITH the exception of a two-mile supplementary trunk line under Sixth Avenue, in the midtown terminal zone of the Borough of Manhattan, the construction of the 55-mile Independent Subway System, owned and financed by the City of New York, is now completed. The Sixth Avenue Line will be built when funds become available for that purpose. At present, trains carrying 600,000 passengers per day are being operated over 33 route miles (Fig. 1). As soon as interior finish work in stations and track installation are completed and operating equipment is obtained, train service will be extended over the remainder of the system. The planning and construction of the system are under the jurisdiction of the Board of Transportation, a department of the city government.

For construction purposes, the subway system, consisting of a number of routes, was divided into contract sections from one-half mile to one mile in length. For each section, contract drawings, specifications, and estimates were prepared, and bids obtained from contractors. Construction work was let on a unit price basis to the lowest bidder on each of the contract sections. The Engineering Department of the Board of Transportation prepared all the plans for the subway structures, inspected the materials required, and supervised all the construction work. To illustrate some of the engineering problems that had to be solved in the planning of the system, two portions, one comprising two and a half route miles of subway along St. Nicholas Avenue in upper Manhattan, and the other, two miles of subway along the Grand Concourse in the Borough of the Bronx, have been selected for description from those parts of the Independent System for which structural plans were prepared under my direction.

ST. NICHOLAS AVENUE LINE

Extending from 123d Street to 160th Street, the St. Nicholas Avenue Line is $2^1/_2$ route miles in length and includes $11^1/_2$ track miles. It was built at a cost of $20,000,000.

St. Nicholas Avenue leads north from the low, flat lands of Harlem, which lie to the south and east of its intersection with 123d Street, to the high plateau forming Washington Heights. Washington Heights, except for Inwood Hill, is the northernmost of the rock ridges and hills in Manhattan and has become a densely populated residential district since the opening of the first subway in 1904. The continuity of St. Nicholas Avenue made it the logical thoroughfare for the new subway connecting Washington Heights and other parts of northern Manhattan with the midtown and downtown business sections.

In the two miles from 123d to 160th streets, the street surface of St. Nicholas Avenue rises 135 ft from elevation 125 to elevation 260, taking mean high water of New York Bay as elevation 100. This datum was adopted to avoid minus elevations in river tunnels. The highest surface elevation along the new subway in Manhattan is two miles farther north, at 190th Street. Here the street surface has an elevation of 350. Beyond 190th Street there is a steep downward slope resulting in a drop of 230 ft to elevation 120 in the Dyckman Valley.

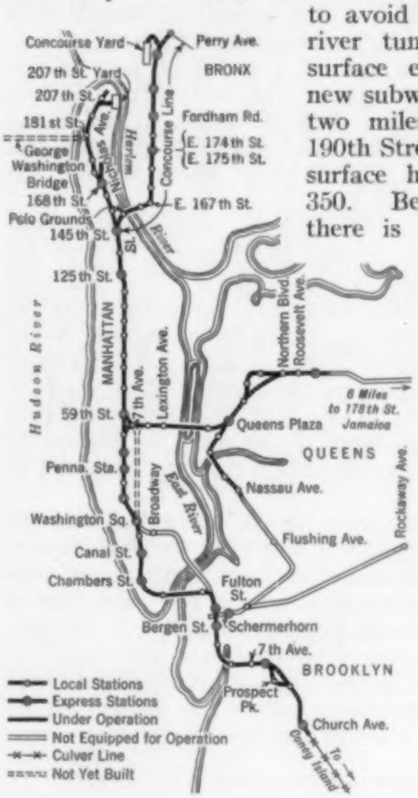


FIG. 1. INDEPENDENT, CITY-OWNED SUBWAY SYSTEM OF NEW YORK

From 123d to 127th Street the subway structure is in filled ground, and the subgrade is below tide level. North from 127th Street to 140th Street, the easterly side wall and most of the track floor is in earth, while the westerly wall in many instances abuts on the 30 to 50-ft rock cliffs of St. Nicholas Park,

From 140th to 160th Street the subway structure is entirely in rock, except for the Polo Grounds Station on the Concourse Branch, which is partly in filled ground.

The St. Nicholas Avenue Subway is a part of the four-track main trunk line of the new city system. Local stations are at 135th Street, at 155th Street, and at the Polo Grounds. Express stations are located at 125th Street and 145th Street, one mile apart. The 125th Street express station is $3^1/_2$ miles north of the last express station in the midtown zone, that at 59th Street and Columbus Circle.

Between 127th and 140th streets, two tracks, each 2,400 ft long, were added to the normal four-track main line to provide storage for trains and also to permit short-line train operation in case of a tie-up on part of the system. North of 140th Street, the four tracks manifold into a seven-track double-deck structure, of which the upper level contains four tracks serving Washington Heights, and the lower level, three tracks serving the Bronx via the Concourse Line.

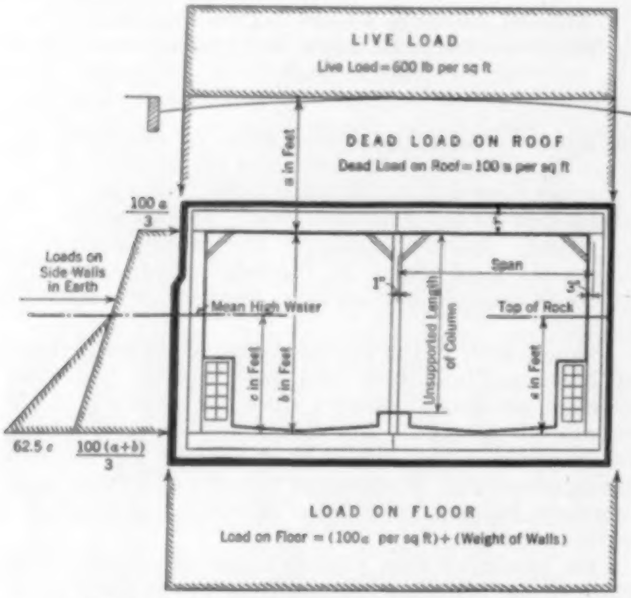


FIG. 2. LOADING DIAGRAM FOR SUBWAY DESIGN

The building lines on St. Nicholas Avenue are irregular because the street reaches Washington Heights by rising along the side slopes of a steep hill. Although St. Nicholas Avenue follows a general northerly direction, there is a total curvature of 86 deg in the subway alignment between 123d and 160th streets. A quarter of the subway structure in these two miles is on curves having radii varying from 500 to 3,000 ft. Between 140th and 160th streets, a distance of one mile, the tracks are on 3 per cent grades except within station limits and at crossovers.

A typical loading diagram for the basic design of the subway is given in Fig. 2. From 123d to 127th Street, where the floor of the structure is in earth below tide-water level, steel I-beams were used in the track floor to spread the superimposed loads uniformly on the soil. The normal 5-ft spacing between bents was varied at 124th Street to allow for the diagonal crossing of a trunk sewer (Fig. 3), which passes under the subway as an inverted siphon, with a storm overflow above the subway.

LAYOUT OF TRACKS COMPLICATES STRUCTURES

At 139th Street begins the first of a number of depressed tracks, which permit splitting the train service of the four-track line from the south between the four-track Washington Heights Line and the three-track Concourse Line. To provide for the depressed tracks, cross-overs, and flexing tracks, which extend for a quarter of a mile to 145th Street, an irregular and complicated structure was required, varying in design with each bent. This called for an unusual amount of detail work in order to obtain a design that would be economical and permit speedy construction.

According to usual practice, the curves used for the track alignment along St. Nicholas Avenue would have required special forms for the roof arches, tapered to suit the divergence of the radial bents. Special form work was reduced to a minimum by using groups of parallel bents with a tapered bay between adjacent groups. The standard adopted was a tapered bay varying in width from 4 ft at the inside side wall to 6 ft at the outside side wall, the number of parallel bents for different groups being varied to fit the adopted taper.

The 145th Street Station is a three-deck structure (Fig. 4) with a maximum depth of 65 ft. The lowest level consists of three tracks and two island platforms for the Concourse Line; the intermediate level, of four tracks and two island platforms for the Washington Heights Line; and the top level, of a mezzanine extending the full length of the station. Within the limits of this station, between 145th and 148th streets, the street surface rises 20 ft. To avoid an unnecessarily heavy backfill on the station roof, the headroom in the mezzanine was progressively increased from 7 ft 6 in. to 10 ft 0 in., and in addition the mezzanine floor was made to slope upward at the north end. This introduced, below the mezzanine, a fourth level containing only struts to brace the steel frame of the structure.

On March 24, 1928, the subway structure of the 145th Street Station was unexpectedly subjected to a severe test. At that time the concrete had been placed to within 60 ft of the north end of the station, and part of the steel frame for the remainder of the station had been erected. The excavation was 65 ft deep and was progressing toward the north in what appeared to be sound rock. However, when it reached 148th Street a mud seam intersecting the side of the cut near the subgrade and sloping up toward the east, caused a wedge-shaped slide, which widened toward the south. The slide was 150 ft long along the easterly side of the excavation, totaled 2,000 cu yd of rock and overlying earth, and extended 20 ft into private property at its southern end. This property was vacant except for a small one-story store building, which was partly wrecked by the slide.

Fortunately, the concreted part of the station structure had sufficient strength and rigidity to stop the progressively widening slide by holding the rock in place. This caused the sliding rock to rotate about the easterly side wall of the completed subway station structure, thereby shearing off a triangular shaped block of rock 50 ft high and 20 ft wide at the top. The slide stopped just short of a group of six and seven-story dwellings on the easterly side of St. Nicholas Avenue. In spite of the seriousness of the accident, it is gratifying to know that a catastrophe was prevented by the completed part of the subway structure and that this structure withstood momentary stresses that could only be exceeded by those due to an earthquake. Between the side wall of the subway and the face of the bank exposed by the slide, eight concrete buttresses were built to brace the rock still in place and also to provide support for future buildings.

North of 149th Street the Washington Heights Line is a double-deck structure in which the two local tracks are on the upper level and the two express tracks on the

lower level. The street surface continues to rise rapidly to the north, and although 3 per cent grades were used where possible, there was room enough above the track roof to construct a mezzanine over the side platform of the local station which extends from 153d to 155th Street. The rising grade of the street and the high level of the rock made tunneling the most economical method of construction from 156th Street to 158th Street (Fig. 5). From 158th to 166th Street the double-deck type of structure was built in cut-and-cover excavation.

BRANCH FOR THE CONCOURSE LINE

At the lowest level of the 145th Street Station the three tracks of the Concourse Line separate from the main line and enter a tunnel 2,000 ft long and from 50 to 110 ft below the surface of St. Nicholas Place. Some of the rock along this tunnel is a disintegrated mica schist which resulted in adverse conditions for tunneling. The thickness of the concrete tunnel lining was varied from 12 in in sound rock to a maximum of 2 ft 6 in. where unstable material was encountered. The tunnel passes under the abutment wall of the 155th Street Viaduct, which is 70 ft high. Here the subway emerges from the rock cliffs of St. Nicholas Place into Manhattan Field, where open-cut construction was used.

At the Polo Grounds a side platform station was built in an open-cut trench 70 ft wide and from 40 to 45 ft deep, the subgrade being 30 ft below the water level. The station structure was designed to carry roof loads of from 3,500 to 5,000 lb per sq ft to provide for the support of future buildings. A structural steel floor was designed for this station to spread the loads uniformly on the sand and clay subsoil.

An open shaft was to be built at the easterly end of the Polo Grounds Station for starting the shields of the three cast-iron lined tunnels crossing under the Harlem River. The bulkhead line of the river is 500 ft east of the shaft location. When the excavation reached a level within 6 ft of the subgrade the bottom began to boil. This situation was met temporarily by dumping 1,500 cu yd of sand into the shaft to maintain the soil in equilibrium.

The shaft was 81 by 42 ft in plan and 50 ft deep, the bottom being 40 ft below high tide in the Harlem River. It was decided to complete the excavation under compressed air, and this required that the open shaft be converted into an air-tight chamber, as shown in Fig. 6. A design had to be developed for enclosing the shaft, taking into consideration the bracing and sheeting already in place, providing for the introduction of shields and locks for tunnel operations, and allowing room for the building of the permanent subway structure. Thus the design involved not merely the final structure but also more than usual consideration of the several stages of construction.

Three sides of the shaft were backed by earth, but the fourth or westerly side was open because it adjoined the then completed excavation and structure of the subway station. A concrete wall with steel framing was built along the open side of the shaft and was braced to the completed station structure. In order to completely enclose the shaft, a temporary roof of steel beams was provided, capable of carrying sand deposited in a number

AFTER 2,000 CU YD OF ROCK HAD SLID AGAINST THE PARTIALLY COMPLETED SUBWAY
On the St. Nicholas Avenue Line

of bins and weighing 2,300 lb per sq ft of roof. For the support of this heavily loaded roof one hundred 14-in. sectional steel shell piles 50 ft long were jacked down to 20 ft below the final subgrade of the excavation. The piles were so placed as to clear the bracing and timbering already in place. Each pile was tested to 75 tons, but the design load used was 50 tons. As the excavation progressed under this roof, the steel sheeting along the sides of the original shaft excavation was lined with concrete. During the excavation to the subgrade of the shaft the maximum air pressure was 12 lb per sq in. At the bottom of the shaft a concrete floor was built in sections to serve both as a seal and as a construction floor. When this floor was completed, timber bents were erected on it, extending from the floor to the roof of the air chambers. The load of the roof was then transferred from the piles to the construction floor through these bents, after which the piles were cut off at the top of the floor. The removal of the piles provided working space in the chamber for assembling the three tunnel shields.

At this stage the compressed air could be dispensed with, since the water pressure on the floor was offset by the weight of the sand on the roof, transmitted through the timber bents. It was then possible to remove parts of the roof, and through the openings thus provided to lower materials and equipment for the shields

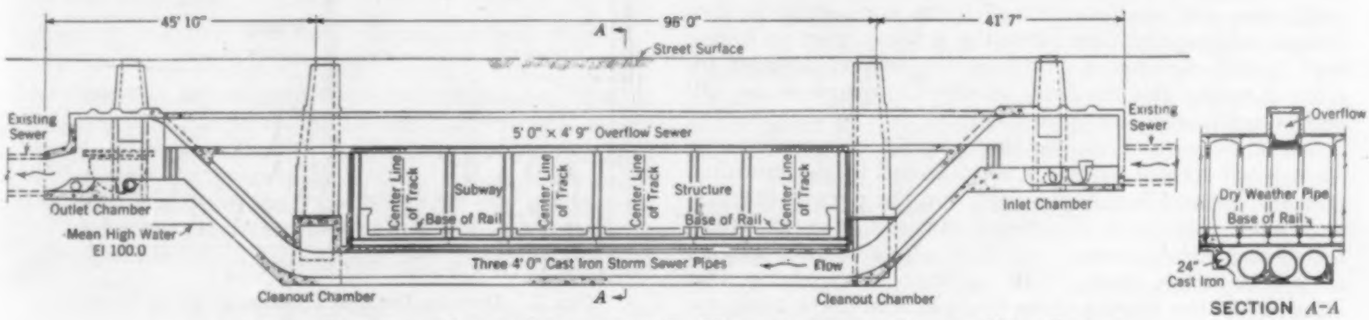


FIG. 3. SIPHON FOR SEWER CROSSING THE ST. NICHOLAS AVENUE LINE AT 124TH STREET

into the chamber. When the roof had been replaced, tunnel operations for crossing the Harlem River were begun under air. On completion of the tunnel, the roof of the shaft chamber was dismantled and the subway structure within the shaft area was completed in the open.

THE CONCOURSE LINE

The Concourse Line enters the Borough of the Bronx at the southerly end of a rock ridge, called High Bridge

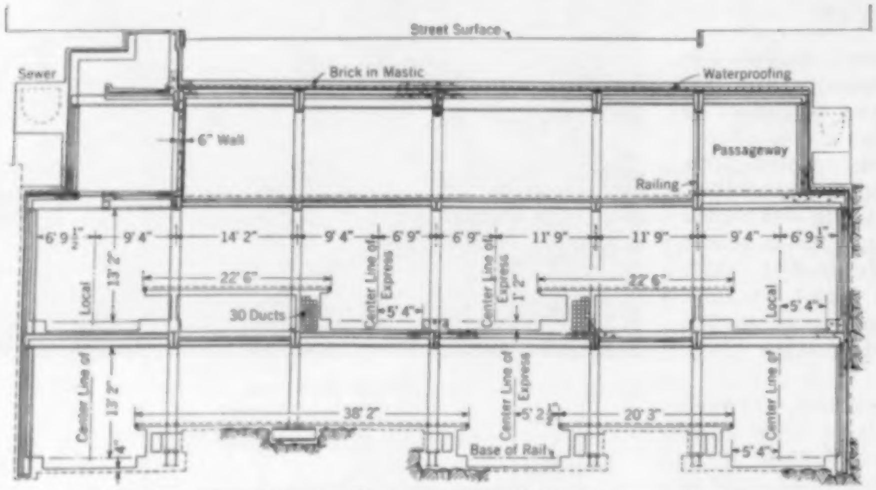


FIG. 4. CROSS SECTION THROUGH THREE-STORY SUBWAY STATION AT 145TH STREET AND ST. NICHOLAS AVENUE

Hill, lying along the east shore of the Harlem River. This ridge begins at 160th Street and extends north to Van Cortlandt Park, and through it to the northerly boundary of the city. The Grand Concourse, which follows the crest of a parallel ridge a half mile to the east, has a total width of 182 ft and three roadways, the center one 60 ft wide, and has become a super-highway for through automobile traffic. It is separated from important crosstown thoroughfares by underpasses.

In locating the Harlem River tunnels of the Concourse Line, it was decided to avoid as much as possible the filled-in ground of Cromwell Creek, which at one time drained the valley between the two ridges. This was accomplished by placing the river tunnels north of what would have been a direct line from Manhattan to the Bronx. A further advantage of this location was that the river tunnels could be terminated in the sound rock of High Bridge Hill. The subway crosses the former bed of Cromwell Creek between Jerome and River avenues, where ground water was encountered, as the subgrade is in clay and silt below tide level. East of River Avenue, the subway rises above tide level and is in firm ground for several hundred feet, after which rock is encountered. Thus, within the limits of the subway station at River Avenue, three different soil conditions—silt-and-clay mud, firm soil, and rock—had to be considered in the design. Along the Concourse the subway is in rock, except in the vicinity of 174th Street, where in order to cross a valley the roadway of the Concourse is on fill between retaining walls 60 ft high.

The Concourse Line has three tracks, the center one for express service, which is southbound in the morning and northbound in the evening. On the part of the line described, there are four local stations, one at 161st Street and River Avenue and three along the Concourse, at 167th, 170th, and 175th streets. Judged by the numbers of the streets, these stations seem to be close together, but they are actually 2,500 ft apart. This section of the line has a length of two miles, a trackage of six miles, and was constructed at a cost of $12,000,000.

Through the River Avenue Station the tracks are on curves of 3,000-ft radius in order to absorb part of the right-angle turn from 161st Street into the Concourse. Beyond the limits of the station, the rest of the angle is turned on curves of 600-ft radius, eased by transitions from 200 to 300 ft long. Curves of varying lengths, having radii from 1,000 to 5,000 ft, were required to follow the irregular line of the Concourse.

From the Harlem River tunnels, the 3 per cent upgrade is continued past the Jerome Avenue portal to a point within Macombs Dam Park, a city park adjacent to the Harlem River, where a minimum cover over the subway roof is attained. In the next 4,000 ft, from the park to 167th Street and the Concourse, the ground surface rises 190 ft, necessitating 3 per cent grades reduced locally to $2^1/_2$ per cent to compensate for the curve at 161st Street. Through the River Avenue Station, a 0.7 per cent grade is used, although normally grades through stations do not exceed $^1/_2$ per cent.

HIGHWAY UNDERPASSES RECONSTRUCTED

To a large extent the planning of the Concourse Line was influenced by highway undercrossings at ten important crosstown thoroughfares. Each of these undercrossings presented a three-level grade-separation problem, the topmost level being occupied by the Concourse, the second or third level by the subway, depending on local conditions, and the remaining level by the underpass. Five of these underpasses occur between 161st and 175th streets.

In order to permit the subway to cross over the 167th Street underpass, the steel frame and concrete structure of the underpass was removed, the grade was lowered a

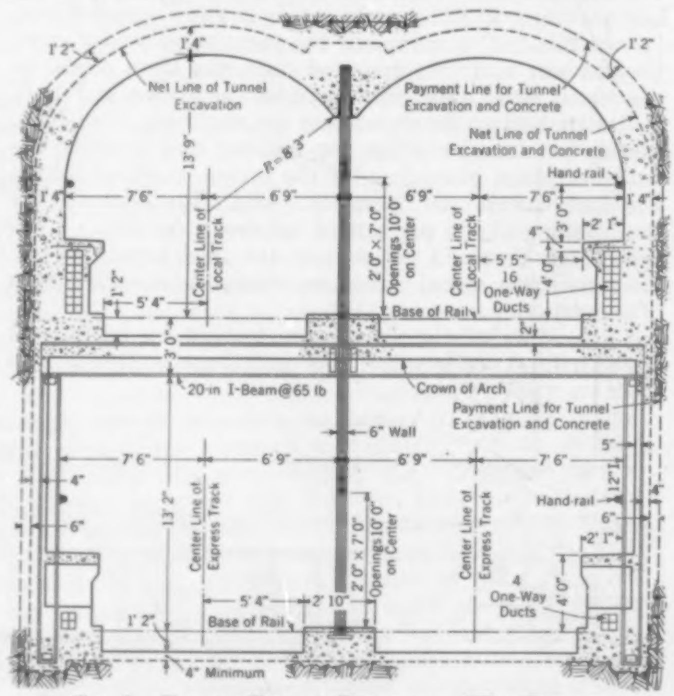


FIG. 5. DOUBLE-DECKED TUNNEL AT 157TH STREET
On the St. Nicholas Avenue Line

maximum of 25 ft, and the structure was rebuilt, except for the retaining walls of the west approach. These walls were underpinned to the new subgrade in a series of pits 4 ft long. A station for surface cars, with two island platforms, each 100 ft long and 8 ft wide, was incorporated in the reconstructed underpass for the convenient transfer of passengers from the surface cars to the subway. Although the new roadway of the 167th Street underpass is 50 ft below the Concourse, it was possible to provide gravity drainage by building 900 ft of sewer to Grand Avenue. This saved the cost of installing and operating pumping equipment.

To bring the 170th Street underpass below the subway station at that point the underpass was lowered 16 ft. It was possible to plan the subway structure so that most of the former roof of the underpass could be retained as part of the roof of the subway station. The expense of shoring the old roof during excavation was warranted, as this saved the cost of removing it and of replacing it by new steel and concrete. At the center of the underpass the roadway was widened so that buses or other vehicles can stop for subway passengers without blocking through traffic. Stairs from the underpass, terminating in the public area of the station, will give bus passengers and pedestrians direct access either to the subway station or to the sidewalks of the Concourse.

At 174th Street the roadway of the underpass is 35 ft below the Concourse and intersects it at an angle of 45 deg. Here the roadway was retained at its original grade, but the headroom was reduced to 14 ft. The original underpass structure was a concrete arch, part of which was removed for the construction of the subway. The subway structure and the Concourse loads above it are carried across the underpass by four Pratt trusses.

To permit the construction of the subway through the underpass at 175th Street, the brick arched roof of the underpass was removed in part. Since the roadway of the underpass was 45 ft below the surface of the Concourse, it was possible to retain the underpass at its original grade and to carry the subway structure through it on girders. The subway station, extending from 174th to 175th Street, has entrances from the sidewalks of the underpasses at both of these streets.

Plans for the underpasses were developed in cooperation with the engineers of the Borough of the Bronx and of the Third Avenue Railway Company in order to meet the conditions created by the combination of subway, highway, and street railway transportation.

SUBWAY STATION SPECIALLY DESIGNED FOR HEAVY TRAFFIC

At the 161st Street and River Avenue Station, special provisions were made to handle the heavy traffic which occasionally comes from the Yankee Stadium, where 80,000 people congregate at times. Space for a large number of turnstiles was provided on the mezzanine, and the station platforms were made 4 ft wider for part of their length so that such crowds could be handled expeditiously.

At the 167th Street subway station, in addition to the usual street entrances, provisions were made for access from the two island platforms of the surface-car station and from the sidewalks of the highway underpass below the subway. The controls on the mezzanine are arranged so that pedestrians can avoid the vehicular traffic on the Concourse by passing under it and through the public area of the subway station mezzanine.

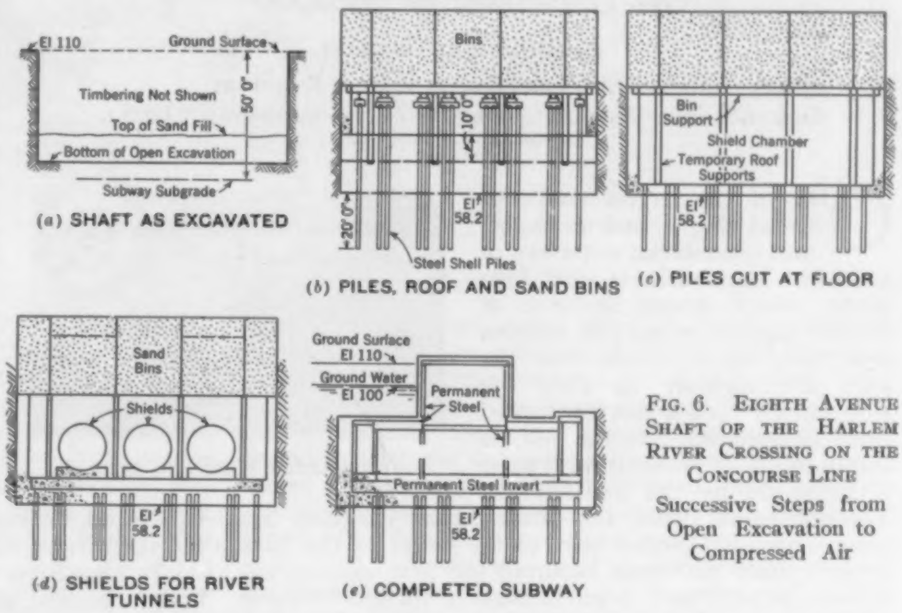


Fig. 6. Eighth Avenue Shaft of the Harlem River Crossing on the Concourse Line

Successive Steps from Open Excavation to Compressed Air

From the easterly end of the River Avenue Station to 165th Street, for a distance of 1,600 ft, a three-track concrete-lined tunnel was built. Except for this tunnel and the Harlem River Crossing, the three-track subway structure of the Concourse Line consists of steel bents on 5-ft centers with concrete jack arches between them.

All the work here described is part of the New York City Independent Subway System, which was planned as the initial step of a coordinated, city-wide development of passenger transportation facilities, intended to initiate an orderly and continuing expansion of subway transportation facilities. With the rapid transit routes of the city augmented by the building of this 55-mile system, it was expected that subway construction could go forward continuously so as to anticipate traffic requirements, to aid in the development of a logical city plan, and to replace obsolete means of transportation by modern rapid transit subways suited to the needs of a growing metropolis. The vastness of the city's urban transportation problem can be visualized from the population growth, which in the three decades between 1900 and 1930 increased from $3^1/_2$ million to 7 million, although New York City's area of 320 sq miles was not enlarged during that time.

The Independent System was built and is being operated by the Board of Transportation of the City of New York. Chairman John H. Delaney and Commissioners Daniel L. Ryan and Francis X. Sullivan comprised the Board of Transportation up to December 1933, when Commissioner Ryan retired and was succeeded by Charles V. Halley. The plans were prepared under the direction of Robert Ridgway, Past-President and Hon. M. Am. Soc. C.E., Chief Engineer (since retired), and A. I. Raisman, M. Am. Soc. C.E., Chief Designing Engineer of the Board of Transportation. The several divisions of the Engineering Department engaged in the planning and construction of these subways were in charge of the following Division Engineers: C. E. Conover, John H. Myers, J. B. Snow, and George L. Lucas, Members Am. Soc. C.E.

Improving the Liao River

Manchurian Stream Diverted by a Stoney-Gate Weir, and Banks Protected by Mattresses

By Bunkichi Okazaki

Member American Society of Civil Engineers
Executive Chief Engineer, Upper Liao River Conservancy Board,
Newchwang, Manchuria

In southern Manchuria (Manchukuo), the Liao River Conservancy Board is undertaking the improvement of the river of that name in the interest of navigation. The work so far completed includes a huge diversion weir and lock and a new river channel 14 miles long. This channel, which has a capacity of about 12,000 cu ft per sec and provides $7^1/_2$ ft of water at low stage, required $2^3/_4$ million cubic yards of excavation. The necessity of building the weir on a foundation of fine, silty material led the engineers to support the structure on wooden piling protected against underleakage with three rows of sheet piles driven to clay. Above and below the weir the river channel is paved with a flexible mattress made of precast concrete blocks strung on cables—a type of channel protection that has been in successful use in Japan for nearly 25 years.

Completed Weir and Lock

One of the largest rivers of China (Fig. 1) and an important commercial waterway of southern Manchuria is the Liao River, which drains an area of 100,000 square miles. It empties into the Gulf of Pohai, near the port of Yingkow. In 1889 the river cut for itself a shorter channel from Tangchiawopu to the gulf, as shown in Fig. 2, probably with some artificial aid by the local people. The new route, called the Shuangtaitzu Channel, has since carried the greater part of the water of the Liao River. Since the break occurred the new channel has become considerably larger because of its shorter route and steeper gradient. In the long and tortuous course of the original river in the first 24 miles below the break, from Tangchiawopu to Sanchaho, there is a hard bottom which resists erosion. At low stages there has been insufficient water to make this section navigable. By 1924, villages had so encroached on this section of the river that it was little more than a ditch, and its rehabilitation seemed out of the question. At Sanchaho, meaning "three river forks," the Liao is joined by the Hun and the Taitzu rivers. The water added by these tributaries, together with the effect of the tides, renders the reach from Sanchaho to the gulf fairly navigable.

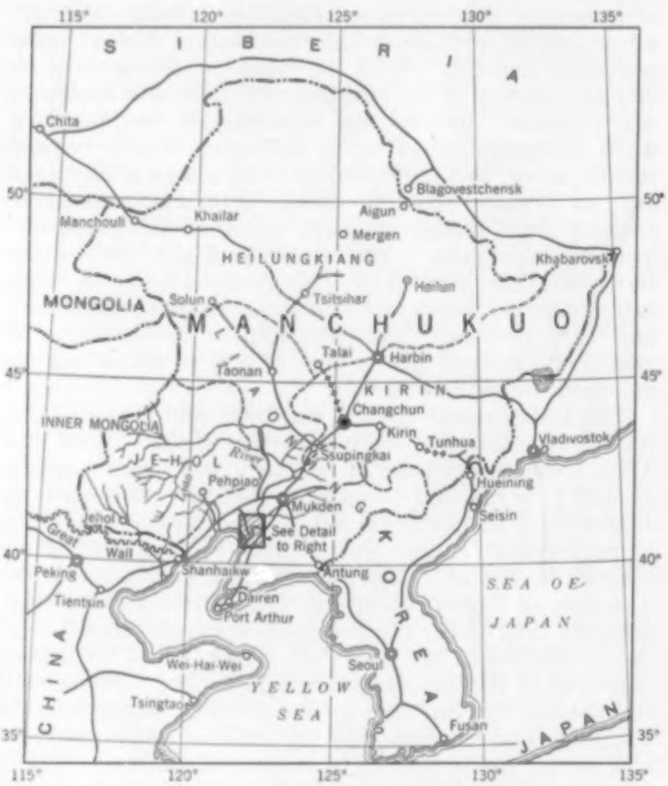


Fig. 1. Liao River Drainage Area

The Liao River Conservancy Board is an international body organized to improve the Liao River. Under my direction a stoney-gate weir and lock were built across the Shuangtaitzu channel at Erhtaochiao to divert a part of the flow into a new canal, nearly 14 miles long, which bypasses the hard-bottomed and narrow section of the upper Liao. The canal, which extends to Chiashintzu, follows the route marked "Line A" in Fig. 2.

Fig. 2. Location of Improvement

With a bottom width of 60 ft and a depth below low water of $7^1/_2$ ft, the new canal has a capacity of 2,600 cu ft per sec at low water and 11,600 cu ft per sec at maximum flood stage. At Erhtaochiao a maximum flood stage of 117.3 ft was recorded during the summer of 1921. At this stage, the flow of the Liao above Tangchiawopu was calculated to be 50,000 cu ft per sec, of which 30,000 cu ft per sec would pass down the Shuangtaitzu channel and 8,000 cu ft per sec down the old Liao River channel. The side slopes of the new cut are 1 on 2; the distance between the new embankments on the inside is 800 ft; and the total excavation is 2,750,000 cu yd.

A short distance downstream from the inlet to the new canal, a weir across the entire width of the Shuangtaitzu Channel was completed in 1929. When closed, its seven sluice gates, of the stoney type, each 37.5 ft wide and 12 ft high, will raise the surface of the river $4^1/_2$ ft above its normal level. The floor of the weir is flush with the river bottom.

Enormous quantities of silt, amounting to 2 per cent

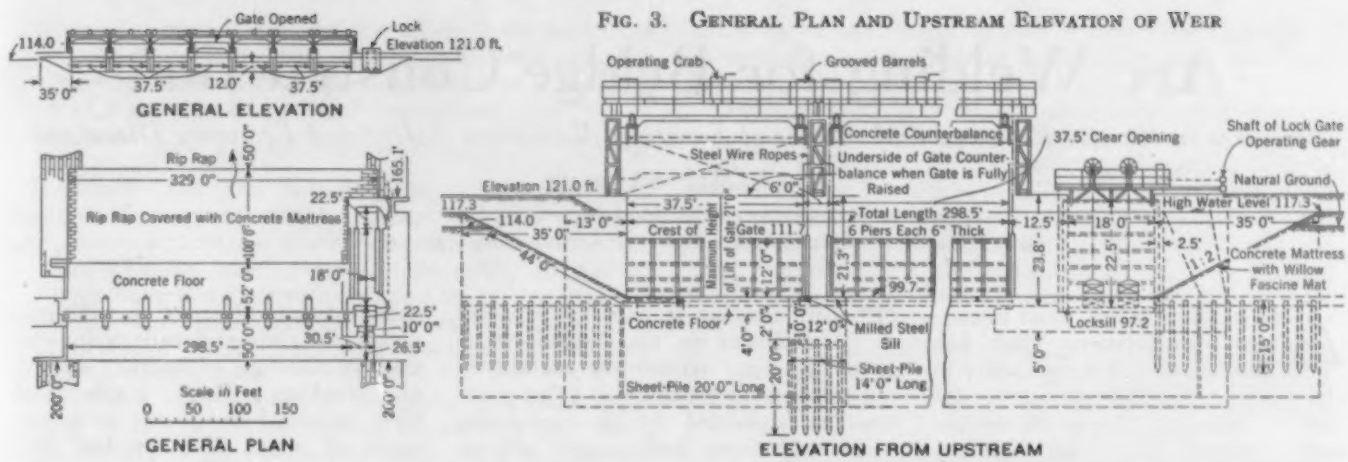


FIG. 3. GENERAL PLAN AND UPSTREAM ELEVATION OF WEIR

by weight, are carried by the river. To prevent the deposition of silt in the new canal or in the channel below the weir, and to scour out the silt that has been deposited, the water surface at the weir is periodically raised by lowering the gates to increase the gradient and the velocity of flow. The gates also provide the means of dividing the water between the old river channel and the new canal. When they are raised above the highest known high-water level the weir does not appreciably contract the river section.

A lock 80 ft long and 18 ft wide has been built alongside the weir to allow junks to pass up and down the Shuangtaitzu Channel at any stage of the river. Details of the design of the weir and lock are given in Figs. 3 and 5, and the completed structure is also shown.

Before construction was started, the river was diverted around the weir. Because of the character of the material on which the weir is built, a very fine sand, the design was predicated on a seepage coefficient of 18. Three rows of concrete sheet piles, 20 ft long, were driven across the structure to hard clay to cut off underflow. The grooves between adjacent sheet piles were grouted to make the cut-off walls thoroughly water-tight. The walls of the weir and lock, and the floor of the lock rest on piles. The concrete floor of the weir, 2 to 4 ft thick, rests directly on the sandy river bottom and is provided with no artificial support.

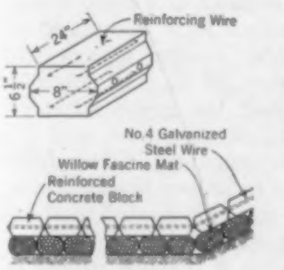


FIG. 4. TYPE OF FLEXIBLE CONCRETE MATTRESS

To provide a solid platform from which to operate the equipment for driving the piles for the walls of the weir and lock, the contractor poured the floor of the structure first. By the time the excavation for the walls was finished and the driving of the piles was completed, the concrete floor had settled in an irregular manner and had seriously cracked. To prevent all possibility of future leakage through the floor as well as to improve its appearance, a thorough job of repair and grouting was undertaken.

All cracks and seams in the floor were enlarged to a width of 6 in. from top to bottom of the slab and then filled with well-tamped concrete. Beginning 2 ft from the foundation of each lock abutment and guide wall, grout pipes 1 in. in diameter were cemented into holes cut through the floor. The spacing between the pipes was about 20 ft in each direction. Ground-water level was drawn down to the bottom of the slab before new concrete was placed in the cracks. After the concrete had thoroughly set, grout under a pressure of 100 lb per sq in. was forced to refusal into 18 of the holes, or until it rose in adjacent pipes. In all, 480 cu ft of grout was used.

In 1909 I developed a type of flexible ferro-concrete mattress for use in Japan. An improvement of this type was adopted for protecting the weir structure on the Liao from undermining and the banks adjacent to it from erosion. Above and below the weir, the bottom of the channel was paved with riprap stones about a cubic foot in size and the side slopes were covered with willow fascines. On top of the riprap and the willow fascine mats a continuous mattress of precast concrete blocks was laid, strung on steel cables like beads on thread (Fig. 4). The blocks are placed horizontally, end to end, and the joints are staggered. The cables run across the stream and up the bank, passing through the left hole of one block, then through the right hole of the next block, thus forming a continuous fabric of blocks. As scouring takes place, the blocks settle to conform to the shape of the new bank or bottom. Increased flexibility of the concrete mattress was obtained by casting the blocks with a hexagonal cross section (Fig. 4). Each block is about 6 by 24 in. in plan and $6^1/_2$ in. thick. Metal molds, with hinged sides so that the block can be released without damage after tamping, were used for casting the blocks. In all, 21,700 sq yd of flexible mattress was built and laid.

All the work described was performed for the Liao River Conservancy Board for a contract sum of $1,277,500, Mexican money, during the period from 1924 to 1929.

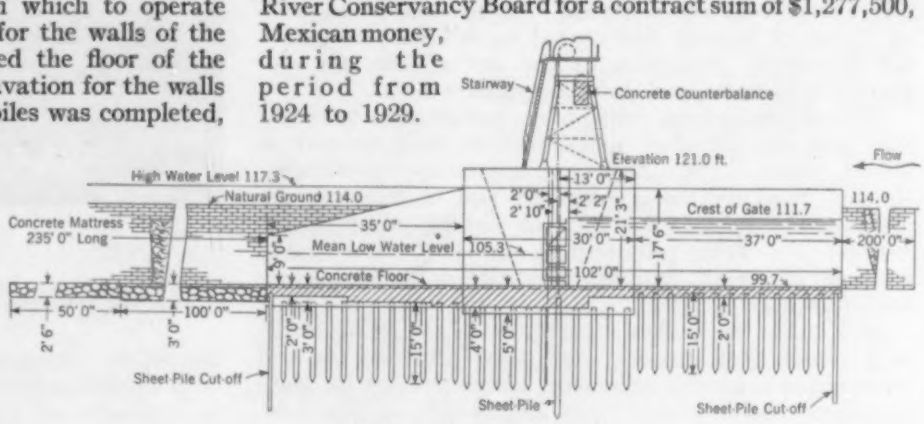


FIG. 5. SECTION THROUGH STONEY GATE DIVERSION WEIR

Arc Welding for Bridge Construction

Progress in the Art Reviewed and Means of Securing Maximum Safety and Economy Discussed

By LAMOTTE GROVER

ASSOCIATE MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
OFFICE BRIDGE ENGINEER, STATE HIGHWAY COMMISSION, TOPEKA, KANS.

THAT welding is rapidly becoming established in the United States as a safe and economical method of construction for bridges and other structures is indicated by the increasing number of municipalities that are revising their building codes to provide for its use and by the number of states which are building, repairing, and strengthening their bridges by this means. The first all-welded bridges to be built in the United States were completed in 1928—a 55-ft plate-girder structure in East Pittsburgh, Pa., and a skew truss at Chicopee Falls, Mass. European engineers have utilized welding for some notable spans, including a 161-ft highway truss in Czechoslovakia and a 150-ft railway bridge in Russia. This article, originally presented before the Kansas State Section on March 17, 1934, contains the results of Mr. Grover's intensive study of the best practices in the art.

ARC welding is the most important development that has taken place for many years in the steel construction industry. Its use in the construction of storage and processing tanks, boilers, piping, naval vessels, boats and barges, vehicle frames and bodies, machinery, and other products has advanced rapidly during the last fifteen years. The progress in its application to structural engineering also has been rapid, although less spectacular. The arc-welding process has been used for the construction of new steel bridges quite widely in foreign countries but seems to have been regarded with indifference by most of the structural engineers of America, who have been content, for the most part, to delegate all judgment regarding the procedure of welding to the foreman of the welding shop, who is often a man of no engineering or technical training.

Recent interest and activity indicate that American structural engineers are finally awakening to the fact that welding has been developed into a dependable and economical method of making joints and connections in steel structures, and further, that it must be applied under proper engineering control if it is to be used with safety, which is predicated only upon the application of the available knowledge of correct design and proper control of procedure. Tests by the American Bureau of Welding of specimens made with bare-wire electrodes showed that with reasonable control, results do not vary more than 12 per cent from the average for a given type of joint. By the use of heavily flux-coated or covered electrodes, the limits of variation have been reduced to about one-half this amount. Throughout this article the term "covered electrode" is used to designate all types of heavily flux-coated or covered electrodes which provide protection to the arc stream, the deposited weld metal, and the adjacent base metal.

Welded connections present less surface area to corrosion and are therefore more durable than riveted or bolted joints. They have the advantage of utilizing the full cross section of members and of reducing the amount of expensive handling of material through the various fabricating processes in the shop. For homogeneous, one-piece, jointless construction, welded structures approach the ideal.

In Kansas the bridge department of the State Highway Commission has for some time provided for and encouraged a steadily increasing use of welding for comparatively minor connections and details in new construction and also for the strengthening, repairing, and widening of existing structures. Often welding has been specified as an alternative, with provisions for riveted details or for cast steel in the case of bearing devices and rockers. These applications have included the flange reinforcement of some 33-in. rolled floor beams in new bridge construction and the shop welding of stiffeners, floor expansion devices, cantilever brackets, hand rails, and diaphragm connections of rolled beam spans. In this way the department has been attempting to gain experience gradually, to train its organization, remove prejudice, and establish a standard of quality among fabricators, all looking toward a more extensive use of welding.

In order to gain further experience in the application of welding to bridge construction, the state highway commissions of both Kansas and Missouri have recently built all-welded girders at the shop of the Kansas City Structural Steel Company and have tested them to destruction. These girders were composed of 12 by 1-in. flange plates with butt-welded splices and 54 by $^1/_2$-in. web plates, and were 27 ft 1 in. each in length. In the Kansas girder half the edges adjacent to the welded joints were planed or machined and the other half were flame cut. Intermittent fillet welds were used for some joints in order to induce high stresses. Also, a number of auxiliary tests were conducted to aid in determining the behavior of the welded joints and connections. There were no evidences of distress in any of the welds. As soon as the results of these tests have been studied and interpreted, they will be made available to those who are interested.

Courtesy A. G. Bissell

ALL-WELDED PLATE-GIRDER RAILWAY BRIDGE, BUILT AT EAST PITTSBURGH IN 1928

For Westinghouse Electric and Manufacturing Company

A number of outstanding all-welded bridges are listed in Table I. In Russia a number of all-welded railway

bridges have been fabricated, including girder and truss spans, one of 150-ft length. The Soviet Government has conducted a number of tests of welding and is building a 150-ft span for testing welded connections.

GENERAL FEATURES OF DESIGN

Naturally, the first tendency in designing welded connections is to adhere too closely to the types that have been commonly used for riveted construction. It cannot be too much stressed that safety as well as economy demands thorough familiarity on the part of the engineer with the fundamentals of the design and construction of welded connections, their possibilities and present limitations.

Care must be exercised in designing structures containing both riveted and welded connections. If both welds and rivets are used in the same joint, the stresses will not be distributed to the rivets according to the usual assumptions because of the greater rigidity of the welds. If some of the fabricating or handling operations involved in the riveted construction of heavy material are not eliminated, the cost of the shop work may be great enough to offset the saving in material that can be effected in the members and their connections by means of welding.

One of the most attractive means of securing economy in bridge construction is the use of welded built-up plate girders in place of riveted girders or rolled beams, because the saving in material is very great and the number of pieces handled is much reduced. Alternative designs were prepared recently by the State Highway Commission of Kansas for a 1,220-ft deck-girder viaduct with a 40-ft roadway and one 5-ft sidewalk. The welded design indicated a saving of 271 tons of steel, or 24 per cent of the 1,136 tons required for riveted construction. This included a saving of 75 tons, or 35 per cent in six spans, for which rolled beam girders were provided in the riveted alternative.

The two most important and distinctive characteristics of welded connections, which must be borne constantly in mind, are their rigidity and the presence of residual stresses caused by the heat of welding. The reason for their greater rigidity is that there is no slip to help equalize stresses. This equalization must all be accomplished by deformation in both the base metal and the weld metal, a fact which indicates the advantage of ductility and toughness in the weld metal and also that of designing for good stress distribution.

Designers should provide symmetry and avoid conditions that will cause warping or create residual stresses due to shrinkage in cooling, especially those acting in the same direction as the principal load stresses. This consideration is most important in details that will be subjected to impact or shock.

Fillet welds are most commonly used in structural work. Butt welds have been avoided by many structural designers in this country because it has been found that the weld metal from bare-wire electrodes is not well adapted to resist shock or repeated stress. An increase in the use of butt welds may be expected with the use of covered electrodes, because butt welds form the ideal joint for good stress distribution.

TABLE I. SOME OUTSTANDING ALL-WELDED BRIDGES

Date	Location	Use	Type	Span	Weight of Steel
1927–1928	Chicopee Falls, Mass.	Railway	Truss	135 ft	80 tons
1928	East Pittsburgh, Pa.	Railway	Plate-girder	55 ft	......
1929	Lowicz, Poland	Highway	Truss	89 ft	55 tons
1930	Leuk, Switzerland	Highway	Truss	122 ft	37 tons
1931	Plzen, Czechoslovakia	Highway	Truss	161 ft	145 tons
1932	Merced, Calif.	Highway	Steel beam	31 at 40 ft; 7 at 20 ft	......
1933	Dresden, Germany	Highway and tramway	Plate-girder	13 at 72 to 86 ft; Total, 1,040 ft	431 tons

Where a choice between end-fillet and side-fillet welds is possible, the former are preferable because they are from 25 to 30 per cent stronger. A combination of end- and side-fillet welds, usually of comparatively small size and placed to completely encircle the connection, or at least along both sides and the end farthest from the joint, is undoubtedly the best practice. If more welding is required for this than is necessary for strength requirements, its use can be justified as a protection from corrosion in unprotected outside structures, such as bridges. With the use of covered electrodes, continuous single-run fillet welds in the smaller sizes, such as $^3/_{16}$ and $^1/_4$ in., are found to be more economical than intermittent welds of larger size. In any case, welds should be continued around corners to avoid abrupt change in section at points of maximum stress concentration.

Courtesy General Electric Company

ARC-WELDED STRUCTURAL STEEL VIADUCT, 1,380 FT LONG, ON ROUTE TO YOSEMITE VALLEY, CALIFORNIA

In designing the details of welded connections, provision must be made for cleats or clip angles temporarily tacked in place, or other means of assuring the proper erection and alignment of members until they are welded. Such devices promote economy by eliminating extra handling of heavy parts or members, which would be necessary if holes were punched in the main members for use as guides during erection; they also permit the full cross sections of the main members to be effective. Enough details for preparation of the material and for the procedure of welding must be definitely specified to provide for the proper construction of the joints.

Studies and experiments have conclusively demonstrated the importance of avoiding notch effects, sharp corners, and abrupt changes in section in order to prevent highly localized stress. There is little doubt that the reinforcement and excessive convexity commonly required by some specifications have actually weakened joints rather than strengthened them, especially when they would be required to withstand impact or repeated stresses.

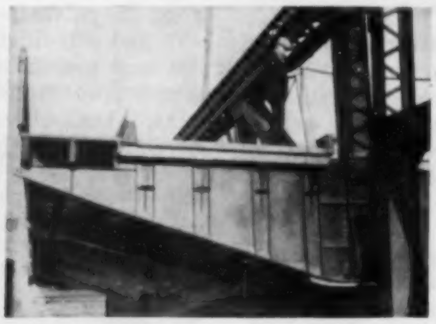
Details should always be arranged so that the operator can see his work and hold the electrode in the proper position.

A mistake likely to be made in designing the details of some kinds of joints is to provide merely the amount of welding indicated by dividing the total stress by the unit stress; and to investigate no further, although the critical section may exist in the base metal.

Unless the fabricating shop is specially equipped for cutting to exact length, the direct welding together of parts of main members, instead of providing connections,

is not usually found to be economical, except possibly at one end of a member.

Most of the provisions of standard specifications are based on the characteristics of welds produced with bare-wire electrodes. Surely a considerable increase in the commonly accepted working stresses is justified solely by the 20 to 50 per cent increase in tensile strength and the 100 to 200 per cent increase in ductility produced by covered electrodes, without taking into account other characteristics which also have been greatly improved.

Courtesy Lincoln Electric Company

REMODELING AND WIDENING UNIVERSITY BASCULE BRIDGE AT SEATTLE, WASH.
Welded Brackets and Stringers for Outer Roadway; One Leaf Raised to Show Welded Roadway Brackets

IMPROVED RESULTS WITH COVERED ELECTRODES

If the working stresses for welds in bridges are held rather low until more experience is gained, it is very important to provide some means of automatically raising them as uncertainty and lack of uniformity are overcome. This could be accomplished by adopting working stresses expressed as a percentage of certain characteristics of welded joints, to be determined from tests of standard types of specimens.

In order to illustrate the improved characteristics of weld metal deposited with covered electrodes according to the most modern procedure, in Table II some typical comparisons are given between such weld metal, mild rolled structural steel, and bare-wire electrode weld metal.

Improved characteristics of weld metal deposited with covered electrodes are partly due to the exclusion of the oxygen and nitrogen of the atmosphere, which cause a lack of uniformity in the composition of the deposited weld metal and an excessive loss of alloying ingredients and form impurities appearing in the form of slag inclusions and gas pockets. They are also due in part to the fact that a heavy slag covering is deposited over the weld metal, further protecting it and the adjacent base metal from atmospheric contamination, annealing them, and keeping the metal in a plastic condition long enough to allow impurities to escape with the aid of the flux incorporated in the covering and long enough to permit residual stresses to be relieved. Covered-electrode weld metal is much more resistant to corrosion than bare-wire weld metal, being equal to mild steel in this respect, as shown by various kinds of accelerated corrosion tests, some made under stresses above the elastic limit to indicate resistance to intergranular corrosion.

TABLE II. CHARACTERISTICS OF VARIOUS WELD METALS COMPARED WITH MILD ROLLED STEEL

CHARACTERISTICS AT 70 F	COVERED-ELECTRODE WELD METAL	MILD ROLLED STEEL	BARE-WIRE OR WASHED-ELECTRODE WELD METAL
Ultimate tensile strength (lb per sq in.)	65,000–85,000	55,000–65,000	40,000–50,000
Yield point (lb per sq in.)	50,000–55,000	30,000–35,000	30,000–32,000
Elongation (percentage in 2 in.)	20–30	22–30	5–10
Reduction in area (percentage)	35–50	30–50	2–5
Impact values, Chapry (ft-lb)	30–35	26–30	5–13
Impact values, Izod (ft-lb)	45–80	40–80	8–15
Endurance value, ten million reversals without failure (lb per sq in.)	28,000–30,000	26,000–28,000	12,000–16,000
Average range of locked-up residual stresses (grain growth) in area adjacent to fusion zone (lb per sq in.)*	3,000–6,000 for slag-shielded and 8,000–25,000 for gas-shielded	....	10,000–35,000
Charpy tension impact values (ft-lb)†	1,250–1,260	....	393–460

* Laboratory of J. D. Adams Manufacturing Company.
† Watertown Arsenal tests of butt-welded joints, 5/8 by 1/2 in. in least cross section.

Another advantage obtained when proper procedure is followed in welding with covered electrodes is that the base metal undergoes very little, if any damage, such as embrittlement. This becomes of more importance as the strength and ductility of the welds approach or exceed the strength and ductility of the base metal.

MATERIALS, ELECTRODES, AND EQUIPMENT

Materials not produced under specifications that have been recognized to ensure good weldability should be carefully investigated. It is hardly possible, however, that poor weldability will be encountered in mild structural steel or cast steel conforming to the standard specifications of the American Society for Testing Materials.

The protection afforded by the covered electrode may be provided solely by means of a slag—preferably of low specific gravity, low surface tension, and low thermal conductivity—which forms as the coating or covering of the electrode melts, and which envelops the drops of molten metal, protects them from the air as they pass across the arc stream, and then forms a full protective and annealing covering over the weld. Or the protection may be provided partly by the creation of a flow of inert or reducing gases, which envelop the arc stream and protect it. In most cases the covering extends slightly beyond the metal core of the electrode in the form of a small inverted crucible which confines the heat of the arc and further protects the vaporized metal at this point.

Inasmuch as the characteristics of welds may be controlled in a number of different ways, by varying the composition of both the electrode core and its flux covering, it is not practical to specify definitely the chemical composition of electrodes in writing structural welding specifications. Satisfactory electrodes can be assured by specifying the minimum requirements for the physical characteristics of the deposited weld metal and welded joints and by indicating the general type and size of electrode. The greatest difficulty that is likely to be encountered in providing for suitable equipment is to prevent the use of the older type of welding machines, which were designed solely for bare-wire electrodes.

Although the procedure for bare-wire welding has been quite well standardized, that for welding with

covered electrodes must vary somewhat according to types and makes of electrodes. A number of special and unusual procedures have been developed by various manufacturers and recommended for use with their particular makes of electrodes. Some of these procedures are not recommended for first-class welding. Also, it may be found that some such procedures require special skill and care on the part of the operator. There are two viewpoints with regard to proper procedure. The first is that the procedure may be varied, depending on the quality of weld desired, which must be consistent with the strength requirements of the joint. The second is, more or less, that a weld is either good or bad—or at least questionable—and that no reliable control is possible unless the best procedure is strictly followed in all cases.

Some welding technicians favor the use of large electrodes, such as the $^5/_{16}$ and $^3/_8$-in. sizes, and of high currents for making large-size welds, except where smaller electrodes are necessary for penetration in the roots of welds and for vertical and overhead work. Their experience and tests indicate that less porosity and slag inclusion occur with such procedure. They also find that when smaller electrodes, such as $^1/_8$ and $^5/_{32}$-in. sizes, and lower currents are used throughout, the weld metal is chilled too rapidly and congeals before full shrinkage in it and in the comparatively slow-cooling adjacent base metal can be offset by creep in the fusion metal. Consequently, higher residual stresses and grain growth occur.

Opinions Differ

According to the experience of others, the use of large electrodes and high currents tends to cause a coarse-grained structure in the adjacent base metal, and the layers or deposits made with such a procedure are too thick to permit thorough annealing of the weld metal and the adjacent base metal by the subsequent deposits and by the slag blanket. A coarse-grained structure causes a decrease in strength, ductility, toughness, and endurance—all of which are essential in bridge connections—and indicates the presence of residual stresses. Also, it is claimed that smaller deposits present a better opportunity for inspection by the operator to make sure that he is getting complete penetration and thorough fusion along all surfaces. For these reasons, some technicians favor the use of lower currents and smaller electrodes, none over $^3/_{16}$ in.

This difference of opinion is no doubt largely due to the fact that each one has drawn broad conclusions from experience with a limited number of types of electrodes, kinds of work, and techniques of welding. The heat-dissipating capacity of the work under various welding procedures is an important factor. A compromise somewhere between the two extremes of procedure probably constitutes the best practice. Of course due allowance must be made by the designer for variations that may be necessary with different types of electrodes and kinds of work.

There is little difference of opinion regarding the procedures for single-run fillet welds in the flat position,

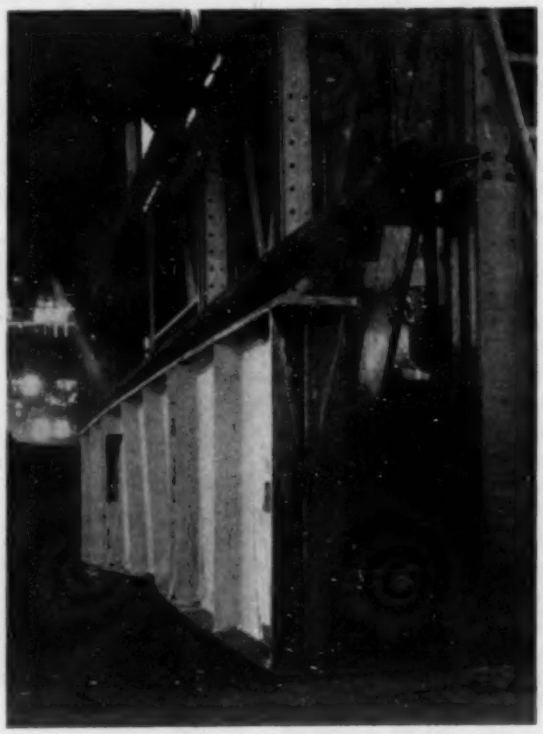
Courtesy Quasi-Arc Company, Ltd.

All-Welded Billingham Beck Bridge, Middlesborough, England

Fillet Welds of $^3/_{16}$ and $^1/_4$ In. Used

which comprise about 90 per cent or more of the welding required on a bridge structure. Undue conservatism should not be permitted to hamper the economic development of the welding process, but the best practices can be employed at the most vital points in a bridge, such as butt-welded girder flange splices, without appreciably affecting the economy of the whole structure.

Welding should be done in the flat position whenever possible because less skill is required and much more speed can be attained. Also, greater strength and ductility can be expected of flat welds, both because the electrodes used for vertical and overhead work carry a lighter coating, which provides less protection and annealing action, and because the metallurgical characteristics of the wire are changed to reduce surface tension and lower the congealing point, all of which necessitate some sacrifice of strength and ductility. In multiple-run welds, each run must be allowed to cool and the slag covering thoroughly removed before the next run is made.



One of the All-Welded Girders Tested to Destruction by the State Highway Commissions of Kansas and Missouri

Typical Manner of Failure, with No Signs of Distress in Any of the Welded Joints

Joints should be welded in such sequence that the members will be free to expand as the work progresses, and the deposition of the weld metal should be balanced and distributed during the welding operation in order to reduce distortion and shrinkage stresses to a minimum. The relief of shrinkage stresses by heat treatment can be accomplished to a limited extent in some types of welded products, but this procedure is impractical for bridge structures. It is doubtful whether true annealing (at temperatures above 1,650 F), to correct grain growth and locked-up stresses, can be accomplished with reliability in any field of practical production. However, it is possible to eliminate the need for

these operations by using the correct welding procedure.

Gas cutting is almost indispensable to the fabricator of welded structures. When performed by skilled operators and with up-to-date equipment, it is surely an acceptable process for preparing mild or structural steel material for welded joints. Where gas-cut surfaces are to be fused to weld metal, it should be required that they be cleaned to remove the oxidized film. This can be done best with power-driven wire brushes.

DROP-TEST MACHINE FOR INVESTIGATING DETAILS OF THE BILLINGHAM BECK BRIDGE

The 2,500-Lb Hammer Struck 69 Blows, with the Drop Gradually Increasing from 2 to 16 Ft. Fracture Started in the Base Metal Some Distance Away from the Weld. Messrs. Dorman, Long and Company, Ltd., Constructors, and Messrs. Mott, Hay and Anderson, Engineers

INSPECTION AND QUALIFICATION TESTS

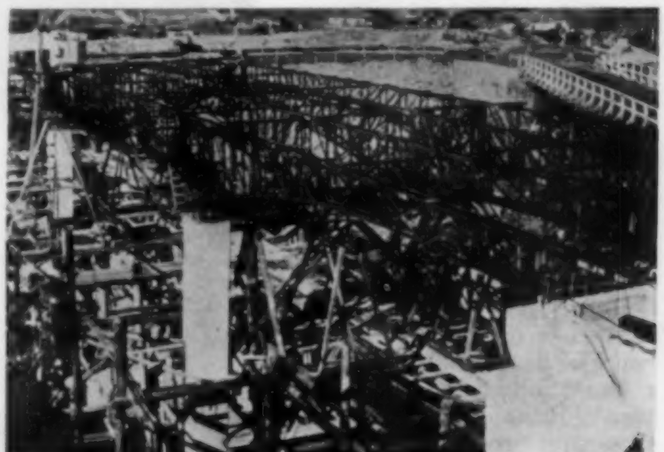
Since so much depends on the skill, experience, and judgment of the welding operators, a great deal of careful attention is merited in making provisions for competent operators and inspectors. A good inspector can determine for the most part whether or not an operator is competent merely by observing his technique and examining the welds as they are being made. Thorough prequalification tests are advisable, however, to supply documentary evidence as to whether or not an operator is competent.

Before operators are allowed to weld important joints and connections they should be required to qualify as prescribed in the stringent Boiler construction Code of the American Society of Mechanical Engineers. Considerations of dependability, which are so important in bridge construction, probably justify this additional expense and trouble. Suitable qualification tests for electrodes are described in the "Tentative Specifications for Filler Metal for Use in Fusion Welding," of June 1, 1933, published by the American Welding Society. In the case of bridge welding, it is desirable to supplement these by tests for impact and fatigue of deposited weld metal and perhaps by tension impact tests of welded joints.

Specificially, progress in structural welding lies in the hands of the engineers who design and build structures. Until they make a thorough study of the details and fundamentals of this type of construction and begin to apply them in actual use, they can expect further progress in welding as applied to bridges and structures to be very slow. It is significant that those who are prejudiced against the use of welding are almost invariably those who know practically nothing about the process or the results of its proper application. Usually they are not reluctant to predict that "it has a great future." This "future," and the resulting increase in the efficiency of steel construction, will never be attained by a policy of laissez-faire.

ACKNOWLEDGMENTS

Acknowledgment is made of the splendid cooperation received from various welding engineers, technicians, and welding operators in discussing problems in a frank and open-minded manner, unbiased by commercial preferences and guided primarily by the desire to disseminate knowledge and assist in placing structural welding on a sound engineering basis. The sources of information consulted were so varied that an attempt to mention them all would probably result in neglect to give credit in some instances where much is due. Special acknowledgment is made of the assistance of E. P. S. Gardner, technical engineer of The Quasi-Arc Company, Ltd., London, England; Robert Notvest, welding engineer of the J. D. Adams Manufacturing Company, Indianapolis, Ind.; M. R. Simpson, of Kansas City, and E. W. P. Smith, of Cleveland, both of the Lincoln Electric Company; and Andrew Vogel, M. Am. Soc. C.E., welding engineer of the General Electric Company, Schenectady, N.Y. All these men rendered valuable assistance by reading the manuscript during its preparation, by making suggestions and furnishing data and information for its improvement, and by supplying illustrations. Interesting illustrations were also sent by A. G. Bissell, consulting engineer, of Seattle, Wash., formerly arc-welding engineer for the Westinghouse Electric and Manufacturing Company.



Courtesy Quasi-Arc Company, Ltd.

WELDED REINFORCEMENT OF REINFORCED CONCRETE HIGHWAY BRIDGE NEAR BALLAM, AUSTRALIA

Heat of Hydration of Cements for Boulder Dam

Methods and Apparatus Used in Tests Conducted by the U. S. Bureau of Reclamation

By Thomas J. Noland, Jr.
Junior, American Society of Civil Engineers
Junior Engineer, U. S. Bureau of Reclamation, Denver, Colo.

SPECIFICATIONS for the cement used in Boulder Dam were prepared only after extensive research. An important aim was to reduce as much as possible the heat generated by the chemical action involved in the hardening of the cement. In order to measure accurately the amount of heat generated, delicate instruments and apparatus were developed. This heat was found to be closely related to the chemical composition of the cement. In this article Mr. Noland gives both a description of the apparatus and technique employed in measuring the heat of hydration and a résumé of the important conclusions drawn from the investigations—a noteworthy addition to the subject.

When concrete hardens, there is a certain amount of heat developed as a result of the chemical reaction between the cement and the water. This is known as the heat of hydration, or heat of hardening, and is usually measured in calories of heat generated per gram of cement. The amount of heat so generated is usually not of great importance in small structures, or in those having a large surface area in proportion to the volume of the concrete. But in massive structures such as the Boulder Dam, the heat of hydration is a factor of considerable importance. In such a structure, the heat is not readily dissipated at the surface, and the temperature rise has a marked effect on the stresses and deformations within the mass.

Heat of hydration is also of importance because it is related to the strength, chemical composition, and other properties of the cement and concrete. Limiting values for heat of hydration were incorporated in the cement specifications for the Boulder Dam and for the Pine Canyon Dam, built by the City of Pasadena. In the future, when massive concrete structures are built, it is probable that the specifications will contain provisions regarding the heat of hydration of the cements used.

It is important to know the rate of heat generation as well as the total amount generated. Heat begins to develop as soon as the concrete is poured, and the rate

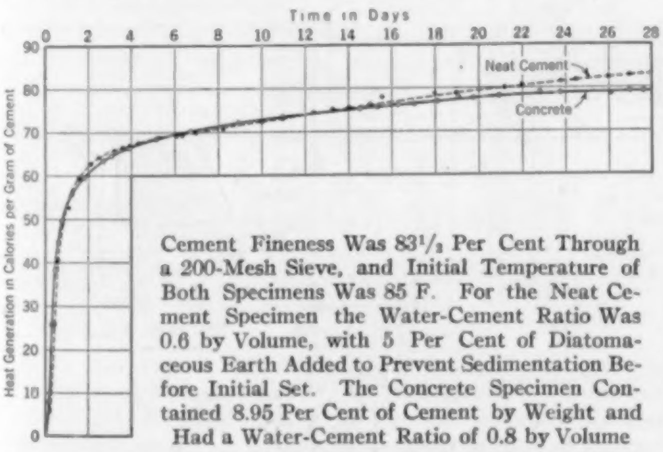


Fig. 1. Typical Time-Heat Curves for Cement and Concrete

at which it is generated is quite rapid during the first week. At 28 days, roughly 90 per cent of the total heat has been developed. At later ages the rate tends to diminish, until after 180 days the slope of the time-heat curve is almost horizontal (Fig. 1).

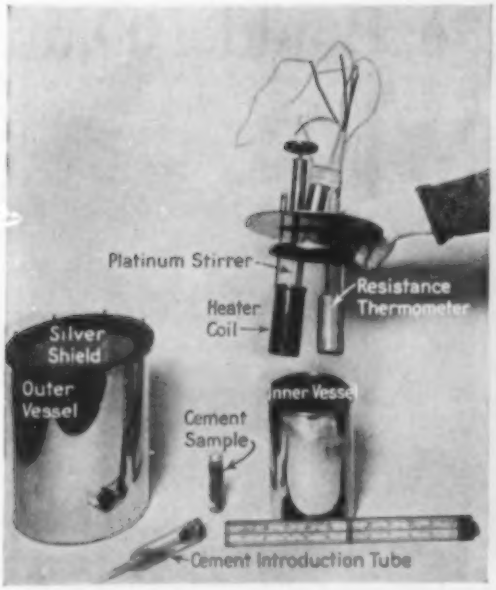
Adiabatic Method of Determining Heat of Hydration

The three methods commonly used for determining the heat of hydration of cement are the adiabatic method, the high-insulation method, and the heat-of-solution method. The adiabatic method is based on the fact that if the temperature of the air which completely surrounds a concrete specimen is regulated so that it is always the same as the temperature within the specimen, there is no loss or gain of heat from the specimen to its surroundings. The temperature rise of the specimen then is the true temperature rise, since no heat is gained or lost. Knowing the number of calories of heat required to raise a unit weight of concrete one degree, and knowing the weight of the specimen and the temperature rise at any age, the total number of calories of heat developed at that age can be computed. If this amount of heat is divided by the weight of the cement in grams, the heat of hydration in calories per gram is obtained.

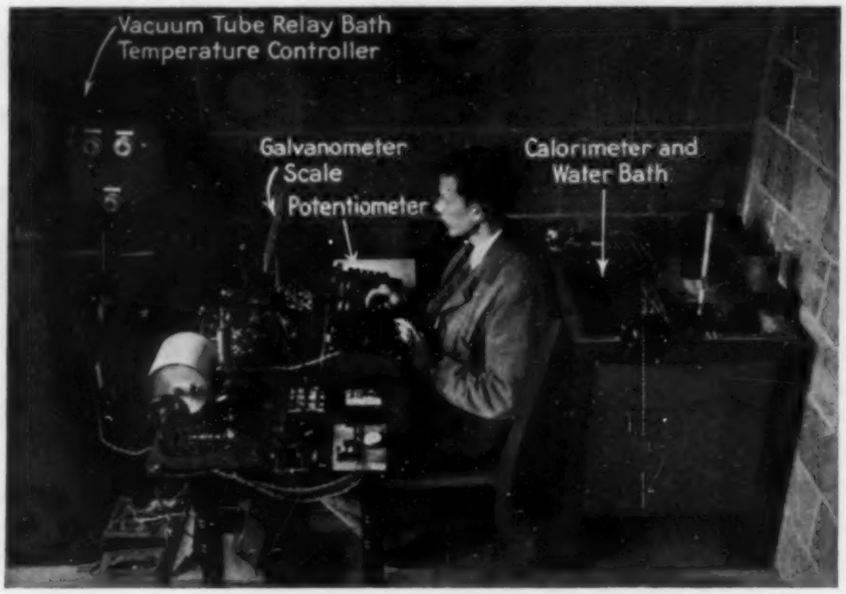
In Fig. 2 the construction details of a typical adiabatic calorimeter are shown. The calorimeter is essentially a heat-insulated compartment into which a large concrete specimen is placed immediately after it is poured. Electric resistance thermometers are embedded in the concrete specimen and are also placed in the air which surrounds the specimen. The temperature of the air in the compartment is regulated so that it is the same as the temperature within the specimen. This is accomplished either manually by an observer or automatically by a temperature recorder. Electric heaters in the compartment raise the temperature of the air, and electric fans circulate the air and equalize the temperature.

High-Insulation Method of Measuring Heat of Hydration

The high-insulation method is based on the same general principle as the adiabatic method. The concrete specimen is placed in a heat-insulated compartment, and the temperature of the air surrounding the specimen is kept constant. Electric resistance thermometers are embedded in the specimen, and the temperature of the specimen is observed at frequent intervals. A heat-transfer correction is applied for the heat lost by the specimen to its surroundings if the specimen is at higher temperature than its surroundings, or for the heat gained by the specimen if the temperature of the



HEAT-OF-SOLUTION CALORIMETER PARTS



HEAT-OF-SOLUTION CALORIMETER AND AUXILIARY EQUIPMENT

specimen is lower than that of its surroundings. This heat-transfer correction is proportional to the difference in temperature between the specimen and its surroundings. The high-insulation method is used mainly to determine the heat of hydration of cements at early ages. It is not sufficiently accurate for use at later ages.

The high-insulation calorimeter is constructed in essentially the same way as the adiabatic calorimeter (Fig. 2). The main difference between the two is that in the former the air temperature is kept constant, whereas in the latter this temperature is regulated so that it is the same as that within the concrete specimen.

HEAT-OF-SOLUTION METHOD OF MEASUREMENT

In the heat-of-solution method, samples of dry, that is, unreacted cement are dissolved in an acid solution in what is called a "heat-of-solution calorimeter," and the heat of solution in calories per gram of cement is computed. Similar tests are made on partially hydrated samples of cement. The difference between the heat of solution of a dry sample and that of a partially hydrated sample of the same cement represents the heat of hydration or the heat liberated by the partially hydrated sample at the age at which it is tested.

A photograph shows the heat-of-solution calorimeter, which consists of an inner gold-lined shell in which is placed a mixture of nitric and hydrofluoric acid used for dissolving the cement. The inner shell is surrounded by an outer shell, from which it is separated by several bakelite plugs and an air gap of about one inch. A very thin convection shield is placed midway between the inner and outer shells to minimize convection currents. The outer shell is fitted with a water-tight cover, and the entire assemblage is placed in a water bath, which is kept at a constant temperature by means of regulating heater lamps controlled by a relay in a vacuum tube circuit. An electric resistance thermometer, a heat coil used in determining the heat capacity of the calorimeter, and a platinum stirrer are placed in the acid solution and are fastened to the water-tight cover of the outer shell.

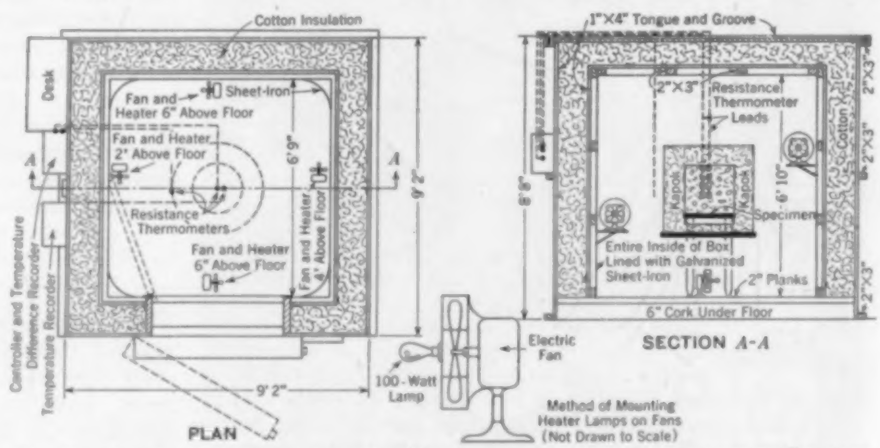


FIG. 2. ADIABATIC CALORIMETER FOR MEASURING HEAT OF HARDENING OF CEMENT
The High-Insulation Calorimeter Is Similar in Construction But the Test Technique Differs

The thermometer used in the heat-of-solution research work for Boulder Dam is one of special design developed in the Engineering Materials Laboratory of the University of California. It consists of four coils—two of manganin and two of copper—wound on the same drum. The coils are arranged in the form of a Wheatstone bridge. A constant electromotive force of about 2 v is impressed across two of the terminals. The electromotive force across the other terminals, which is a function of the temperature, is measured by a precise potentiometer. The thermometer will detect temperature differences of 0.0001 C and is accurate throughout its range to about 0.0005 C.

To illustrate the use of this method, suppose that a dry cement sample is dissolved in an acid solution and is found to have a heat of solution of 580.3 g-cal (calories per gram of cement). A number of cement-paste samples are made, using the proper water-cement ratio, and are put in small glass vials. The vials, in turn, are placed in variable-temperature curing chests, in which the samples are cured under temperatures which approximate those that will obtain in the actual struc-

ture. At each age for which the heat of hydration is desired, one of these samples is removed from the curing chest, the glass removed, and the sample ground and tested in the acid solution.

Suppose that the heat of solution of a 7-day-old hydrated sample is found to be 521.7 g-cal. Then the difference between the heat of solution of the dry cement, 580.3, and the heat of solution of the partially hydrated sample, 521.7, gives 58.6 g-cal, which represents the heat of hydration of the cement at 7 days.

If a sample is taken out of the curing chest and tested when it is 28 days old, it will be found that the heat of solution is less than that obtained for the 7-day-old sample because some additional heat has been liberated between 7 and 28 days. In the same way, at each later age the heat of solution of the hydrated sample becomes less, and the difference between this heat and that of the unreacted sample, that is, the heat of hydration, increases. If values of heat of hydration are plotted against time, a curve is obtained that is substantially the same as that secured by the adiabatic or other methods. Neat cement samples are used in the heat-of-solution method in place of concrete, since the aggregate in the cement is inert and does not contribute appreciably to the heat of the solution.

The heat-of-solution method furnishes the most rapid and economical means for measuring the amount of heat generated. By it a test can usually be made in one or two hours. The labor required for mixing the small neat cement samples is negligible when compared with that required to mix the large concrete specimens used in other methods.

Another advantage of the heat-of-solution method is that it takes into account, and includes, the initial heat of hydration. When other methods are used, about an hour of time usually elapses before the specimens can be put in place, the resistance thermometers connected, and conditions brought to equilibrium. The amount of heat generated during this time is lost and cannot be measured. The heat-of-solution method was first employed in cement research by the Research Department of the Riverside Cement Company, of Riverside, Calif.

MAJOR RESULTS OF THE HEAT-OF-HYDRATION STUDIES

Among the more important facts revealed by the heat-of-hydration studies are:

1. The chemical composition of cement has a marked effect on the amount of the heat of hydration. It is possible to reduce the heat of hydration below that of normal portland cement by proper regulation of the chemical composition.
2. In general, at 28 days, under conditions of mass curing, each 1 per cent of tri-calcium-aluminate produces approximately 2.5 g-cal; each 1 per cent of tri-calcium-sulfate produces approximately 1.1 g-cal; and each 1 per cent of di-calcium-sulfate and tetra-calcium-alumino-ferrite produces approximately 0.45 g-cal.
3. Increasing the fineness of the cement increases the heat of hydration at the early ages.
4. Increasing the water-cement ratio increases the heat of hydration at the early ages.
5. Cements that show a great deal of strength in proportion to the amount of heat generated are those which are low in tri-calcium-aluminate.
6. Heat of hydration is increased by lowering the initial temperature of curing.

The heat-of-hydration studies here described were carried out by the U. S. Bureau of Reclamation at the University of California. They formed part of an extensive program of research on cement and concrete for the project. In this program, 88 commercial and laboratory cements were studied, mainly for the purpose of collecting data to be used in determining the best cement for the dam. Tests were made for strength, volume change, durability, water absorption, and consistency, as well as for heat generation. In addition, a number of minor tests were included. In this research program, which extended over a period of two years, a total of about 30 men were employed.

ENGINEERS' NOTEBOOK

From everyday experience engineers gather a store of knowledge on which they depend for growth as individuals and as a profession. This department, designed to contain practical or ingenious suggestions from engineers both young and old, should prove helpful in the solution of many troublesome problems.

Sheeting for Underpinning Pits

By Howard F. Peckworth, Assoc. M. Am. Soc. C.E.

Ridgewood, N.J.

In carrying out underpinning operations for the Independent Subway along Schermerhorn Street and Lafayette Avenue, Brooklyn, several thousand feet of pits were sunk. The resulting observations constitute a very fair estimate of the speed at which such pits can be sunk and the approximate cost per foot.

These pits varied from 3 by 4 ft to 6 by 6 ft in inside dimensions. The conventional method was abandoned in favor of the patented dovetail method shown in Fig. 1, the 2 by 8-in. timber sheeting being notched beforehand, lowered into the pit, and set in place as the pit was dug. The normal procedure was to dig down along one side of the pit a distance of 8 in. below the bottom board and smooth off the damp sand so that there was a clear vertical face of sand 8 in. by the width of the pit. A notched board was set in place against this vertical face and then forced snugly against the sand by being tapped with a hammer to ensure a close fit. The opposite board was put in similarly, and after that the boards on the two remaining sides were gently forced into place, making a snug fit. By this method pits as large as 6 by 6 ft were sunk through damp sand 65 ft to water without the slightest difficulty, the only precaution necessary being to see that no void spaces were left in the sand behind the boards when they were forced into place.

Typical data obtained while sinking these pits are contained in Table I, which is made up from observations made by me while working as principal assistant to J. C.

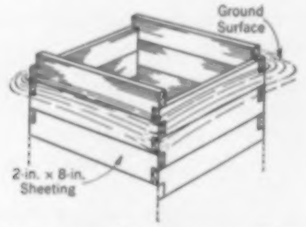


FIG. 1. METHOD OF DOVETAILING SHEETING FOR UNDERPINNING PITS

Meem, M. Am. Soc. C.E., consulting engineer. Sheeting was made of 2 by 8-in. material which cost \$35 per thousand fbm. At locations A and B, referred to in Table I, the rates of pay for 8 hr were: foreman, \$13.33; timberman, \$10.00; and helper, \$5.00. At the remaining

TABLE I. OBSERVED COST DATA ON EXCAVATING UNDERPINNING PITS

Location	No. of Pits	Average Depth of Pits, Ft	Size of Pits, Ft	Rates of Sinking Pits, in Ft per Hr	Labor Cost per Ft	Material Cost per Ft	Labor and Material	Description
A	25	30	6 × 6	0.4975	\$4.80	\$1.80	\$6.60	Difficult digging, in hard clay with gravel and boulders
B	3	18	3 × 4	0.7941	3.69	1.10	4.79	Excellent digging, in coarse brown sand
C	8	43	3 × 5	0.9385	3.42	1.25	4.67	Excellent digging, in coarse brown sand
D	22	31	4 × 4	1.3266	2.97	1.25	4.22	Excellent digging, in coarse brown sand
E	9	19.4	4 × 4	0.5980	3.03	1.25	4.28	Excellent digging in coarse brown sand
F	5	18.5	4 × 5	0.6446	3.83	1.38	5.21	Excellent digging, in coarse brown sand
G	1	8	4 × 5	0.8750	3.18	1.38	4.56	Excellent digging, in coarse brown sand

locations the rates of pay were: foreman, \$10.00; timberman, \$6.40; and helper, \$4.80. An underpinning gang consisted of a foreman and twice as many helpers as timbermen. Neither insurance nor profit is included in the cost of labor and materials given. This work was performed by the Del Balso Construction Corporation, with John R. Monaghan, M. Am. Soc. C.E., chief engineer, and J. C. Meem, M. Am. Soc. C.E., consulting engineer.

Ideal Running Speed for Pelton Wheels

By CHESLEY J. POSEY, JUN. AM. SOC. C.E.

INSTRUCTOR OF MECHANICS AND HYDRAULICS, STATE UNIVERSITY OF IOWA, IOWA CITY

AS usually stated, the ideal peripheral speed for the buckets of a Pelton wheel is that which will cause the water to leave the wheel in a path perpendicular to the path of the buckets. The following analysis shows that this is not generally true, that when friction is neglected the velocity of exit should have a component in the direction of rotation of the buckets, and that when friction is considered the velocity of exit may have a component in either direction.

A cross section of a Pelton wheel bucket is shown in Fig. 1. The jet of water is not turned through an angle of 180 deg, but leaves the bucket at an angle α, to permit the next bucket to clear. Velocity diagrams for the point where the jet enters the bucket and the point where it leaves the bucket are shown in Fig. 2. Vector v_1 represents the absolute velocity of the water in the jet, and is a measure of the total energy available. Vector r represents the velocity of the water relative to the bucket. As the water follows around the curve of the bucket, the magnitude of this velocity is decreased by friction, so that at the point of exit the velocity of the water relative to the bucket is represented by kr, where k is a coefficient less than 1.00.

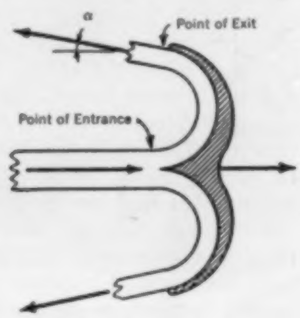


FIG. 1. SECTION THROUGH A PELTON WHEEL BUCKET

The vector v_2 represents the absolute velocity of the water leaving the bucket, and is a measure of the part of the available energy that cannot be utilized in the wheel. The best running speed for the wheel is that which will reduce v_2 to a minimum, v_1 remaining constant. As will be shown, this does not give the condition of jet discharge perpendicular to the runner, either when frictional loss in the bucket is neglected or when it is considered. Neglecting frictional loss, k is equal to 1.0, and from the velocity diagrams,

$$v_1 = b + r$$

and

$$v_2^2 = b^2 + r^2 - 2br \cos\alpha$$

Remembering that v_2 is to be a minimum, while v_1 is held

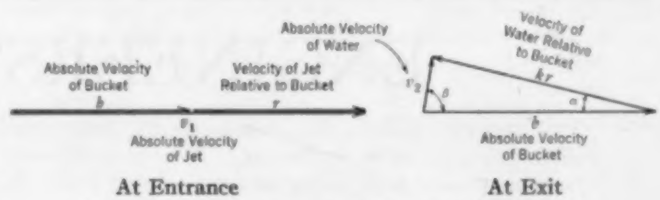


FIG. 2. DIAGRAMS FOR THE VELOCITY OF THE JET

constant, it is easily shown by differential calculus that for greatest efficiency,

$$b = \frac{v_1}{2} = r \quad . \; . \; . \; . \; . \; . \; . \quad [1]$$

The discharge is not perpendicular to the runner but at an angle β, as shown in Fig. 2, such that

$$\beta = 90° - \frac{\alpha}{2}$$

Considering frictional loss, and referring again to the velocity diagrams, Fig. 2,

$$v_1 = b + r$$

and

$$v_2^2 = b^2 + k^2r^2 - 2\,kbr \cos\alpha$$

For greatest efficiency v_2 should be a minimum while v_1 is held constant, which gives

$$b = \frac{k^2 + k \cos \alpha}{1 + k^2 + 2k \cos \alpha} v_1 = C\, v_1. \quad [2]$$

In Table I are given values of the coefficient of v_1 for different values of k and α.

TABLE I. VALUES OF C, IN EQUATION 2, FOR ORDINARY VALUES OF α AND k

VALUES OF α	$k = 0.70$	$k = 0.80$	$k = 0.90$	$k = 1.00$
$\alpha = 8°$	0.411	0.444	0.474	0.500
$\alpha = 16°$	0.410	0.443	0.473	0.500
$\alpha = 24°$	0.408	0.442	0.472	0.500

In this analysis the variation of windage losses and of k with change of speed has been neglected. The windage loss increases as the bucket speed increases, while losses due to friction and impact on the bucket, included in k, decrease as the bucket speed increases. It seems reasonable to expect, therefore, that the ideal velocity relations developed in the present analysis will agree closely with experimental results.

Pavement of Granite-Faced, Precast Blocks

By LOUIS F. HEWETT, M. AM. SOC. C.E.
CHIEF ENGINEER, FREDBURN CONSTRUCTION CORPORATION, NEW YORK, N.Y.

A NEW type of non-skid permanent pavement has recently been constructed on the Albany Post Road, between Ossining and Scarborough, N.Y., where there is a constant flow of heavy trucks and other traffic. The pavement consists of 12 by 12 by 5-in. precast concrete blocks with a granite face or top $2^1/_2$ in. in depth. The blocks were laid on a screeded 1-in. dry mortar cushion over a hard subgrade, which was previously cleaned. The blocks were placed by hand in rows, with $^1/_4$-in. joints, and the joints broken in alternate courses. The pavement was tamped or rolled and the joints filled with asphalt. No skilled labor was used in the work.

In manufacture, the blocks were cast with the top face downward. Granite chunks, each having a surface area of about 5 sq in. and a depth of $2^1/_2$ in., were placed by hand in the form, covering the entire face of the form, and the interstices were filled with 1:1 grout. The remainder of the form was filled with $1:1^1/_2:3$ concrete. The form was then vibrated to increase the density of the final product. In the laboratory the concrete tested up to 7,000 lb per sq in. in compression, and a block failed at 10,000 lb per sq in.

At this point the Albany Post Road is 34 ft wide and has a center section 20 ft wide, paved with brick laid on a concrete foundation. Two concrete shoulders about 7 ft in width flank each side of the brick roadway. The work was done in the following steps. Half of the roadway was shut off, about 12 ft of the old brick surface was removed, and the edge of the concrete shoulder was cleaned. Then the precast blocks were placed on a screeded cushion in rows perpendicular to the concrete shoulders, meeting the surface of the shoulders as closely as possible. After half of the road was completed it was covered with a thin layer of sand and immediately opened to traffic. Then the other half was barricaded and paved similarly. The new pavement was layed as fast as the old roadway was removed and was opened immediately to traffic.

This experimental section of the Albany Post Road was paved by the maintenance forces of the New York

Laying the Blocks

Pouring the Joints

Pavement Half Finished

LAYING A PRECAST, GRANITE-BLOCK PAVEMENT
Experimental Road Laid on a Dry Mortar Cushion Over a Hard Subgrade

State Department of Highways. The cost of the labor for paving was less than 20 cents per sq yd, including the mixing and screeding of the cushion, the carrying and laying of the blocks, and the pouring of the joints. The asphalt filler cost 7 cents per sq yd, and the mortar cushion, 13 cents per sq yd. This pavement, known as "Ultimate," has the advantages inherent to a granite-block pavement at about two-thirds the first cost.

To determine the cost of repairing this type of pavement, the engineer had six rows of blocks removed from the completed roadway and replaced by new blocks. The asphalt which clung to the removed blocks, when knocked off and melted up, was used again. It took one man 40 min to complete the repair, and no visible trace of a patch remained. This repair and replacement cost 8 cents per sq yd for labor.

Our Readers Say—

In Comment on Papers, Society Affairs, and Related Professional Interests

Locomotive Acceleration by Graphical Calculus

DEAR SIR: In his article, "Problems in Locomotive Acceleration," in the April issue, Mr. Barrow presents a method of finding time and space to accelerate a train by a mathematical solution. To get top speed in going up a grade, he gives two methods of obtaining the answer to the cubic equation, one of which is graphical. Another graphical method of finding the top speed is to use the net tractive power curve, keeping in mind the fact that for a $^1/_2$ per cent grade the 10-lb line intersects the curve at 18.1 mph, as indicated by the dotted lines in Fig. 1. This speed agrees with the dynamometer readings instead of the curve from the calculated equation; the calculated curve is in error a little at this point. However, any of the three values, 17.5, 17.6, or 18.1 mph, is close enough.

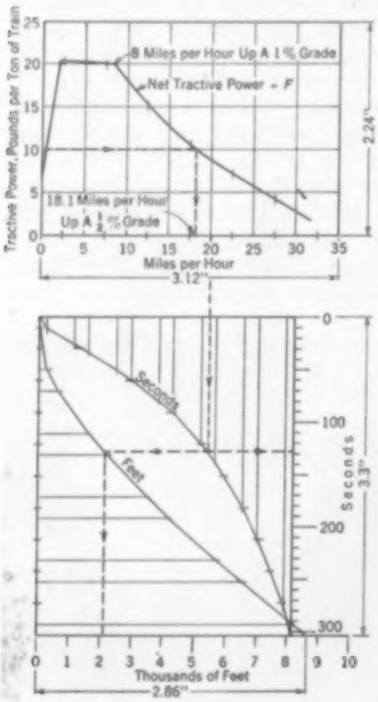


FIG. 1. A GRAPHICAL SOLUTION OF LOCOMOTIVE ACCELERATION

The integral of the reciprocal of the net tractive power curve gives time of travel in seconds, and the integral of the second-velocity curve gives distance of travel in feet. These values come from the following equations, $t = \int (1/a)dv$, and $s = \int vdt$, where $a = F/m = Fg/W$.

The net power curve used in the accompanying Fig. 1 was drawn from the scaled dimensions of Mr. Barrow's Fig. 1, from which the following scales are obtained. The velocity scale is $35 \times 1.467/3.12$ in. = 16.45 ft per sec per in. The $1/a$ scale is $W/gF = 2{,}000 \times 2.24$ in./$32 \times 25 = 5.6$ per in. Therefore the t scale or the $1/av$ scale is $5.6 \times 16.45 = 92.2$. The space scale, s, or the vt scale, is $16.45 \times 92.2 = 1{,}515$. The time required to reach a speed of 30 mph is then 3.3 in. $\times$ 92.2 = 304 sec, and the space traveled is 2.86 in. $\times 2 \times 1{,}515 = 8{,}680$ ft.

By marking these lengths in seconds and feet, the time and distance required to reach any speed up to 30 mph are given by these curves. As shown by the dotted lines, at 20 mph t equals 128 sec and s equals approximately 2,100 ft.

Although the graphical integration can be accomplished by the method shown in my article on page 216 of the April issue, this problem was solved with the Jacob integraph instrument.

BRENT C. JACOB, M. Am. Soc. C.E.
Electrical and Mechanical Engineer
Industrial Brownhoist Corporation

Cleveland, Ohio
May 31, 1934

Existence of Helicoidal Flow

TO THE EDITOR: A mathematical analysis of the mechanics of flow around bends, such as that developed by Lieutenants Blue, Herbert, and Lancefield in the May issue, indicates that a tendency toward helicoidal flow must exist in every river bend. However, in many natural rivers the ratio of depth to width is small, and the forces tending to produce helicoidal flow are overcome by the complicated forces resulting from bed and bank friction.

In our article on "Flow in River Bends," in the May 1933 issue of CIVIL ENGINEERING, the experiments reported were made on a slightly distorted model of the Mississippi River, and in that particular model it was definitely determined that helicoidal flow did not exist, although bed material was moved consistently from the concave to the convex side. Subsequent experiments conducted at the U. S. Waterways Experiment Station with highly distorted models, that is, models having a comparatively large depth-to-width ratio, have indicated that helicoidal flow undoubtedly exists. The conclusion follows that in the model of low distortion the forces tending to produce helicoidal flow were overcome; whereas in the models of high distortion these forces were predominant.

The same reasoning can be applied to natural streams. Thus, in a river such as the lower Mississippi, with a depth-to-width ratio of, say, 1:200, pure helicoidal flow may not exist; whereas in a stream such as the Iowa River, with a depth-to-width ratio of, say, 1:40, this type of flow may be in evidence. We erred in failing to develop this important point in our original article, and Lieutenants Blue, Herbert, and Lancefield have performed a valuable service in presenting data that have caused the point to be brought to attention.

The statement by the authors that "more weight can properly be given to results obtained on an actual river bend" cannot be taken in toto. Causes and effects can be observed, measured, and segregated in a model to an extent far beyond what is possible on an actual stream. Consider, for example, some of the data presented by the authors. The longitudinal profiles of the water surface, as presented in Fig. 3, show that there exists no superelevation (and hence no transverse slopes) at Sections 1 and 4; whereas the diagram of velocity vectors (Fig. 2) shows that helicoidal flow exists at both these sections. These facts are inconsistent with the mathematics presented by the authors, and no explanation is offered. Were such an anomaly to occur in the data from a model study, it is entirely probable that its causes would be readily determinable.

In our article, in the May 1933 issue, we stated that in unnatural bends, where caving had been resisted by a bank difficult of erosion, the sharpness might be so pronounced as to produce higher velocities on the convex side. In this special case bed materials would be deposited on the concave side, while corresponding caving and deeper water would be found on the convex side. The experiments of Lieutenants Blue, Herbert, and Lancefield bear out this conclusion and offer a splendid case in point in showing how the river has changed its course from 1925 to the present. It should also be noted that higher velocities in each instance are found in the locations of greatest depths.

HERBERT D. VOGEL, Assoc. M. Am. Soc. C.E. *and*
PAUL W. THOMPSON, Jun. Am. Soc. C.E.
First Lieutenants, Corps of Engineers, U. S. Army

Vicksburg, Miss.
June 6, 1934

Tests on Plate Action

TO THE EDITOR: The behavior of the test panel described by Mr. Sandberg in his article, "What Is Plate Action?" in the May issue, is confirmed by other tests, which indicate a tendency for the corners of freely supported slabs to deflect upward. Actual raising of the corners occurs only when the supports are of great stiffness and are consequently subject to comparatively small deflections. Shallow supporting beams may deflect sufficiently to cause slab and beams to remain in contact throughout. In an extreme case, when the slab is carried by flexible steel beams, there may be concentration of load near the corners and lowering of the beam away from the slab at mid-span.

Using the data given in Mr. Sandberg's article, I find that a 10-in. 30-lb I-beam is a suitable supporting member. The deflections for this beam relative to the slab are such that the beam and slab remain in contact throughout.

Torsional resistance of mutually perpendicular strips, through which a large part of "plate action" may be justified, is fully effective only when the sides of the slab remain straight, that is, when supports are rigid and corners are anchored against uplift. These conditions are assumed in analyses based on the theory of elasticity. When corners are not anchored, indications are that about 50 per cent of the "plate action" is destroyed. Flexibility of supports tends to increase this percentage.

It may be noted that the relative effect of torsional resistance diminishes with an increase in the conditions of end restraint. Also, negative moments at restrained edges are not decreased by torsional resistance to the same extent as are positive moments.

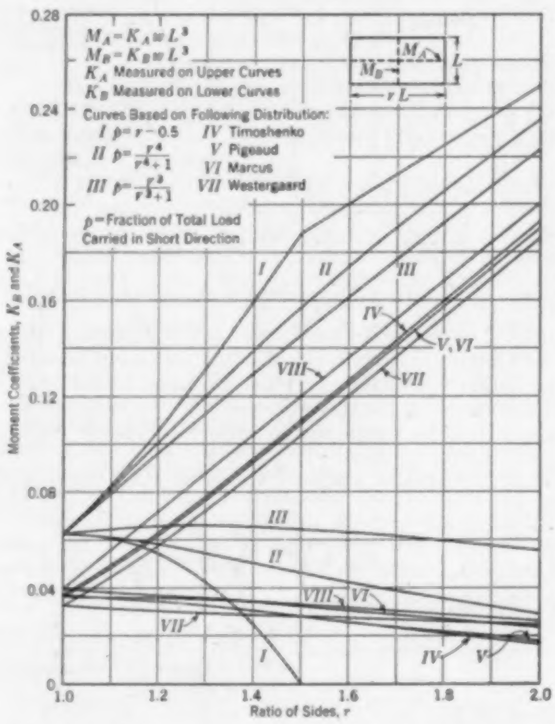


FIG. 1. BENDING MOMENT COEFFICIENTS, FREELY SUPPORTED SLABS

Requirements of existing building codes make no allowance for the strengthening effect of "plate action." The curves in the accompanying Fig. 1 give a comparison of bending moments as calculated by current building code specifications (Curves I, II, and III) and by more precise methods (Curves IV, V, VI, and VII). The more precise curves are grouped together and are distinctly apart from the others. As a basis for a safe and economical method of design, a straight line might be used, such as the one numbered VIII. This line provides for a considerable margin of safety over the more precise curves and allows for uncertainties in theoretical development and for possible deflection of supporting beams. The following equations for freely supported panels result from this line.

$$M_A = 0.16(r - 0.75)wL^3 \quad . \; . \; . \; . \; . \; . \; . \quad [1]$$

$$M_B = 0.014(3.86 - r)wL^3 \quad . \; . \; . \; . \; . \; . \quad [2]$$

in which M_A = maximum total bending moment on section parallel to long side (ft-lb)
M_B = maximum total bending moment on section parallel to short side (ft-lb)
r = ratio of long side to short side
L = length of short side (ft)
w = panel load (lb per sq ft)

Whereas the effect of "plate action" in the slab is to reduce bending and to make possible the use of a thinner section, the effect on the supporting beams is of the opposite character. With the corners not anchored, there will be concentration of loading in the middle region of the beam equal and opposite to the slab reactions shown in Fig. 2 (*a*). With the corners held down and the supports rigid enough to remain nearly straight, the shearing

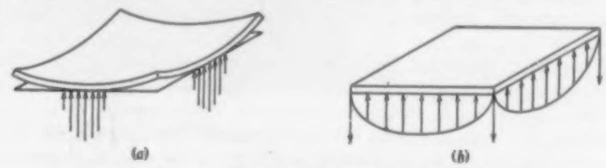


FIG. 2. DISTRIBUTION OF SUPPORTING FORCES

stresses along the periphery of the slab vary roughly as the ordinates to an ellipse. The beam loading will have a similar variation and will be equal and opposite to the slab reactions indicated in Fig. 2 (*b*). The total beam load will exceed the total slab load by the amount of the downward forces at the corners. With either condition the required size for the supporting beams is greater than would be the case if the effect of torsion were omitted.

E. MIRABELLI, Assoc. M. Am. Soc. C.E.
Assistant Professor of Structural Design
Massachusetts Institute of Technology

Cambridge, Mass.
May 25, 1934

Coordinated Planning

TO THE EDITOR: The symposium on the "Advantages of National Planning," in the March issue, is so ably presented that there can be no question as to the desirability of various phases of such planning as advocated by the authors of the papers. However, a phase of national planning that deserves greater attention than it has so far received is local planning, which is sympathetically regarded as a somewhat independent, useful function rather than as an integral part of national planning.

There are several disadvantages in the present lack of coordination between these two kinds of planning. First of all, the development of central planning in advance of local planning deprives it of the intelligent and organized support of the people at large. The second disadvantage in the unbalanced development of the two activities is that the more rapid growth of national planning causes an undue centralization of all the activities affected. This

is apt to create antagonism among the localities, which suffer from the unwieldiness of the national plan as well as from its apparently arbitrary rule.

Still another disadvantage is in the natural limitation of a centralized planning system, which must be general and outside the individual human problems affected by its activities, although the welfare of the people is its ultimate object. For example, if it is within the province of the national planning body to decide on the elimination of a certain number of miles of railroad, who will plan how to meet the problems immediately created by the reduced assessed valuation and reduced employment in the locality affected, if there is no local planning?

The difficulties that the national recovery program is beginning to encounter invariably can be traced back to the lack of understanding and organized support of the country at large. Since national planning is the most important means of overcoming the present chaos, it should be carefully established throughout the country, and every precaution should be taken to avoid the rise of criticism and resentment that might endanger the success of the entire program.

The national planning organization should be developed by taking into account economic units rather than political subdivisions, except in the case of the state unit. If the National Planning Board limits itself to stimulating local planning, it is bound to develop too much along the lines of existing obsolete political units. Thus it is the responsibility of the National Planning Board not only to stimulate local planning but also to organize a complete national unit, made up, let us say, of five regional units. These units should be divided into state units, and the states into units of trading areas, all working toward a definite goal.

In the national system the minimum unit should be at least a center of population with its entire trading area, rather than a city, or a city with its metropolitan area. This would prevent the development of selfish home-town planning as opposed to the larger idea of national planning. At the same time, it would open the way for the eventual reorganization of local government to make it consistent with modern economic life, as mentioned in Mr. James's article.

If we delay stressing the importance of local planning until experimental data from the Tennessee Valley Authority are obtained, it may be too late. The simultaneous development of central planning and local planning by the National Planning Board, with the cooperation of local groups, is a matter of urgent necessity. The responsibility for the proper organization of a national planning system rests with the National Planning Board. However, much can and should be done by local groups to ensure the success of this planning as a new national function.

SERGEI N. GRIMM, Assoc. M. Am. Soc. C.E.
Consulting Engineer

Syracuse, N.Y.
May 29, 1934

Analytical Determination of Fixed Points

TO THE EDITOR: The graphical determination of the fixed points presented by Mr. Gottschalk, in the May issue, is based on the assumptions that the far ends of the adjoining beams and columns are fixed and that the moduli of elasticity of the materials of which the beams and columns are constructed are the same.

The same results may be obtained analytically by the application of the following equations:

$$e_B = \frac{K_B - k_a}{3\left(K_B - \dfrac{k_a}{2}\right)} \quad \ldots\ldots\ldots\ldots [1]$$

$$e_A = \frac{K_A - k_a}{3\left(K_A - \dfrac{k_a}{2}\right)} \quad \ldots\ldots\ldots\ldots [2]$$

in which k_a is the stiffness factor of the beam under consideration and the other values, those used by the author.

If the far ends of the adjoining beams and columns are free, Equations 1 and 2 become

$$e_B = \frac{K_B - k_a}{3\left(K_B - \dfrac{k_a}{3}\right)} \quad \ldots\ldots\ldots\ldots [3]$$

$$e_A = \frac{K_A - k_a}{3\left(K_A - \dfrac{k_a}{3}\right)} \quad \ldots\ldots\ldots\ldots [4]$$

These equations suggest another graphical solution, shown in the accompanying Fig. 1, which I consider simpler and more accurate from the standpoint of geometrical construction.

TABLE I. THEORETICAL VALUES OF e_B AND e_A

Restraint of Far Ends		When $E_c = E_b$		When $E_c = 1.5E_b$	
Columns	Beams	e_A	e_B	e_A	e_B
Free	Free	0.269 l	0.224 l	0.279 l	0.246 l
Free	Fixed	0.276 l	0.230 l	0.284 l	0.250 l
Fixed	Free	0.275 l	0.239 l	0.286 l	0.260 l
Fixed	Fixed	0.282 l	0.245 l	0.291 l	0.263 l
Gottschalk's Method		0.282 l	0.245 l	0.282 l	0.245 l

Based on the example illustrated by Mr. Gottschalk, the theoretical values of e_B and e_A are given in Table I for different combinations of far end restraint of the adjoining beams and columns for $E_c = E_b$ and $E_c = 1.5\,E_b$, in which E_c is the modulus of elasticity of the column and E_b, that of the beam.

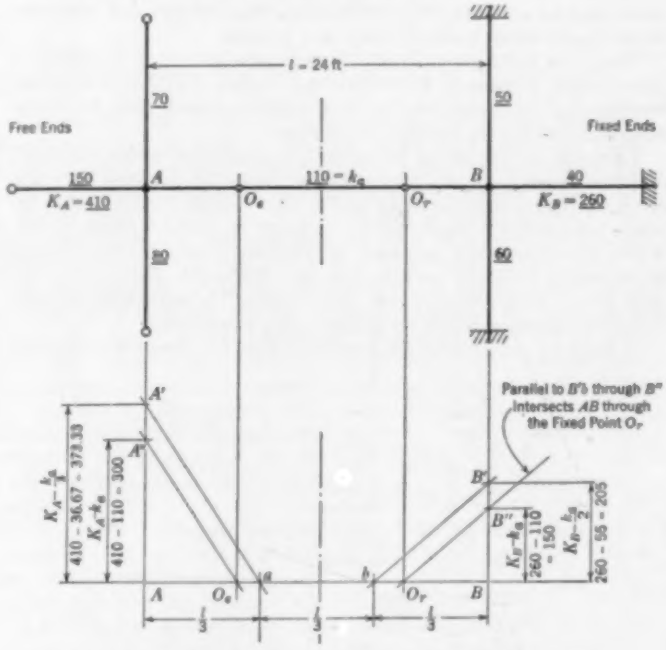


FIG. 1. GRAPHIC DETERMINATION OF FIXED POINTS

This tabulation is given for the purpose of indicating the variations between the extreme range of actual values and those derived by the author's method. It must be remembered that these values are based on the assumptions that the supports have no width and that the values of e increase with that of the ratio of width of support to span.

I am of the opinion that the values of k are more correctly defined as

$$k = \frac{IE}{l \text{ or } h} \text{ than as } \frac{I}{l \text{ or } h}$$

as given by Mr. Gottschalk.

R. L. BERTIN, M. Am. Soc. C.E.
Chief Engineer, White Construction Company, Inc.

New York, N.Y.
June 1, 1934

SOCIETY AFFAIRS

Official and Semi-Official

Annual Convention at Vancouver, B.C.

Not since 1925 has the Society met in official session outside the United States. In that year the Fall Meeting was held in Montreal, Canada. Our neighbors to the north, however, have not been infrequent hosts. The records show an Annual Convention in Ottawa in 1913, another in 1897 at Quebec, and another in 1881 at Montreal. There are many members who recall with pleasure the hospitality dispensed by the engineers of Canada.

Now again a Canadian city, Vancouver, B.C., has honored the Society by inviting it to hold this year's Annual Convention within its confines. The date, July 11–14, has been announced, and a detailed program of the general and technical sessions has appeared in the June issue. At this convention the Society will be the special guest of the Engineering Institute of Canada, and the general sessions will be held jointly with those of the Institute's Western Professional Meeting.

Subjects of special interest in connection with the stupendous engineering developments taking place in the Northwest will be presented, such as the following: the development of the Columbia River for irrigation, navigation, and power; the great central valley development in California; the Fort Peck Dam and Reservoir on the Missouri River; the proposed highway from the United States through British Columbia and the Yukon to Alaska; the Inter-America Highway, proposed to traverse and connect the two western continents; the recently completed Going-to-the-Sun Highway over Logan Pass in the Rockies; and the latest developments and improvements in treating water, disposing of municipal refuse, and salvaging sewage in the West.

Considering the Annual Convention as a raison d'être for a visit to British Columbia, many possibilities for interesting trips present themselves. On three large water storage and regulation projects progress has reached a point where they might well be visited either en route to or from Vancouver—the Fort Peck Dam on the Missouri River, the Bonneville Dam on the Columbia River, and Boulder Dam on the Colorado River. Railroad fares are low, and travelers from Chicago and more eastern points can return via southern California without paying extra fare. This routing will permit a stopover at Las Vegas, Nev., for a side trip to Boulder Canyon. Then too, many other more purely recreational side trips can be taken, including a boat trip to Alaska up the scenic Inside Passage; visits to Yellowstone, Glacier, and Jasper National Parks; to Mt. Rainier, in Washington; to Crater Lake, in Oregon; to the Yosemite Valley, in California; to the Grand Canyon, in Arizona; to Zion Canyon, in Utah; and to many other scenic spots too numerous for mention here.

Vancouver offers many recreational opportunities—for fishing, hunting, yachting, and golf. The fisherman will be interested to know that the waters surrounding the city abound in game fish, both salmon and trout, and that arrangements can be made for this sport either in the bay or the fresh-water streams nearby. The illustration on the Page of Special Interest at the beginning of this issue is suggestive of some visitor at the Convention tasting the joys peculiar to his favorite sport in a setting of great natural beauty. Here also is a yachtsman's paradise. Boats of every description are available and the points to which trips can be made are multiple and varied.

Indications are that a representative group of engineers, many accompanied by their families, will gather at Vancouver on July 11 to derive the inspiration that friendship and common interests always afford. Here is an outstanding opportunity to concentrate vacation plans on a trip that will serve the purposes of professional advancement, social contacts, and travel through country world famous for its beauty.

Carl Ewald Grunsky, Past-President, 1855–1934

After a long and distinguished career in his chosen profession, Carl Ewald Grunsky, President of the Society in 1924, died in San Francisco on June 9. He was born on April 4, 1855, near Stockton, Calif., where he received his early education, graduating from the high school there in 1870. Ambitious for a medical career, he went to Germany to study but changed to engineering and entered the Polytechnic Institute at Stuttgart. There he earned the degree of Diplom. Engr. in 1877. In 1910 this same institution awarded him the degree of Dr. Ing. Later, in 1924, Rensselaer Polytechnic Institute bestowed on him the degree of Eng. D.

The Late Carl Ewald Grunsky
President of the Society in 1926

He returned to California from Germany in 1878 and entered the service of the state, beginning as a topographer on river surveys. He served the state for ten years, rising step by step as the excellence of his work was recognized, until he became assistant and finally chief assistant state engineer, in which capacity he dealt mainly with flood control, river, irrigation, and other hydraulic problems. From 1886 to 1899 he maintained a consulting office, first in Sacramento and then in San Francisco, handling sewerage and water supply projects and a number of irrigation developments. He was called in to report on the sewerage system for San Francisco, and later was a member of a board of engineers who designed the system.

In 1900 he was elected city engineer of this city and during his four-year term, in addition to handling the routine work of this office, he began the investigation of available sources for the domestic water supply of the city, recommending an aqueduct from the Tuolumne River. Moreover, he designed a project for its progressive development, including a distribution system. However, these plans were not the ones accepted when, several years later, the project was started. During his incumbency he also designed a gas works and a telephone system for the city and reconstructed the Geary Street cable railway and extension as an underground electric conduit system.

In 1904 President Theodore Roosevelt appointed him to the Isthmian Canal Commission to complete the surveys for the Canal. From 1905 to 1907 he advised the then Secretary of the Interior, Ethan A. Hitchcock, and was consulting engineer to the U. S. Reclamation Service on irrigation matters. In 1908 he reopened his consulting office in San Francisco and until his death maintained it there.

During his private practice he served as consulting engineer to the Imperial Irrigation District, the Sacramento Public Utility District, the East Bay Municipal Utilities District, and the Santa

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Clara Conservancy District. He also served as a member of the Board of Review of the Sanitary District of Chicago and reported on bridge clearance requirements over the Columbia River below Portland, Ore.

As president of the California Academy of Science, he aided in the establishment of the famous museum and aquarium in Golden Gate Park, San Francisco. In 1930 he was elected president of the American Engineering Council.

During all his crowded professional life, Dr. Grunsky found time to devote to the work of the Society. He became a Member in 1898 and contributed freely to its publications. In 1910 he received the Norman Medal of the Society, the highest award for papers judged worthy of merit as a contribution to engineering science. This was for his paper, "The Sewer System of San Francisco, and a Solution of the Storm Water Flow Problem." From 1919 to 1921 he served as Director from the District in which San Francisco is located. In 1922 he was elected Vice-President of the Society, and in 1924, President.

He was a frequent contributor to the technical press and published several books, among them *Topographic Stadia Surveying; Valuation, Depreciation, and the Rate Base;* and *Public Utility Rate Fixing*.

In 1884 Dr. Grunsky married Mattie Kate Powers of Sacramento. Two sons and two daughters survive him. Both the sons followed their father's profession and both are Members of the Society. Mrs. Grunsky died in 1921.

Keeping the Membership List Correct

IN EXAMINING certain miscellaneous publications of the Society, a few copies of which are kept in stock, a thin pamphlet came to hand, which proved to be a mid-year supplement of the annual List of Members for 1880. It has the following introductory note:

"The following list contains all additions in the various classes of Society Membership which have been made since the issue of the last printed list of July 1880. This list also contains the present addresses of all members whose addresses have been changed from the printed list of July 1880. By substituting these addresses for those in the printed list, and by making the additions here noted, the list will be complete to February 1, 1881. The names and addresses below are printed on but one side of the paper so as to be pasted into the printed list if desired."

In the six months covered by this list of 54 years ago, there had been but 24 additions to the membership, 10 deductions, by resignation or by death, and changes of address for 46 members. The total membership of the Society at that time was less than seven hundred. It was then the custom to publish in each monthly issue of PROCEEDINGS such information as was collected in this pamphlet. Although the additions, transfers, and deductions from all membership grades are still published, now in CIVIL ENGINEERING, one can only imagine how impossible it would be to show in the same way all the changes of address that are constantly being received and registered in the files, affecting a membership of 15,326 as of June 9, 1934.

The card index system at Headquarters, containing the latest address, professional connection, and grade of membership of each member, is revised daily as the result of information gleaned from correspondence. It is desirable for members to notify the Secretary promptly of any change in their address or business connection.

New Secretary for United Engineering Trustees, Inc.

AT ITS MEETING on May 25, the Board of Trustees of the United Engineering Trustees, Inc., elected John Arms to be secretary to succeed Alfred D. Flinn, M. Am. Soc. C.E., whose resignation was accepted at the same meeting. Since December 9, 1933, Mr. Arms has been general manager of the corporation and he will continue to serve in that capacity, combining the duties of both offices. He will be the chief administrative officer for the Board of Trustees in caring for the Engineering Societies Building, the Engineering Societies Library, and the endowment funds which the corporation holds for the American Society of Civil Engineers, The American Institute of Mining and Metallurgical Engineers, The American Society of Mechanical Engineers, and the American Institute of Electrical Engineers jointly. These properties are valued at approximately $4,000,000. Certain changes of organization and procedure necessitated by this change of personnel having been completed by the Board of Trustees at its meeting on June 22, the new secretary assumed his full duties and responsibilities on June 25.

A graduate of Cornell Univeristy, Mr. Arms is a member of The American Society of Mechanical Engineers. His professional specialty has been industrial engineering. He was born in Williamsport, Pa., in 1888, and now resides in East Orange, N. J.

The resignation of Mr. Flinn as secretary of the corporation, regretfully accepted due to conditions beyond his control, terminates the joint secretaryship, which has existed for seventeen and a half years, during which he has served with conspicuous faithfulness. He will continue to be the director and secretary of The Engineering Foundation.

A member of the American Institute of Mining and Metallurgical Engineers, as well as of the Society, Mr. Flinn was elected in 1917 joint secretary of the United Engineering Trustees, Inc. (then the United Engineering Society), The Engineering Foundation, and the Engineering Council, and entered upon his duties on January 1, 1918. Previously he was Deputy Chief Engineer of the Board of Water Supply of the City of New York, Catskill Aqueduct. When the Engineering Council was discontinued at the end of 1920 to make way for the Federated American Engineering Societies, Mr. Flinn was elected part-time vice-chairman and then chairman of the Division of Engineering of the National Research Council. This connection ended in February 1923 because the Engineering Foundation, having in 1922 elected him as its director, as well as secretary, requested all his time, excepting the quarter engaged by the United Engineering Trustees, Inc.

New Home for Providence Section

FOR YEARS the Providence Section of the Society has been affiliated locally with the Providence Engineering Society, and through this connection it is enjoying the prospect of a new home under excellent surroundings. This fortunate condition is the outcome of the recent cash purchase by the Providence Engineering Society of its own building.

While the structure was built for the special use of the Telephone Company, it is admirably fitted for its new tenants. Three stories in height, 30 by 90 ft in size, and of substantial brick construction, it has all the necessary characteristics. Besides ample space for a lounge and library, a clear auditorium for two hundred people is available. In addition, the ground floor of the building will be rented, probably as doctors' offices, more than defraying the operating expenses.

The new building is centrally located, within about a half mile of the center of the city, although in a residential district. It is one block from the Brown University Campus and about two blocks from the College Engineering Building, where many of the special meetings of the Providence Engineering Society are held.

Local Sections of many of the national societies are also benefiting jointly. The experience of the Providence Section and the Providence Engineering Society is most encouraging as indicating the advantages of sound financing and good judgment applied to an engineering organization.

Bringing Index for TRANSACTIONS *Up to Date*

BECAUSE PROCEEDINGS is not issued during the months of June and July, it might be supposed that there is a general lag in the Society's office work. This is far from the case. The fact is that, if anything, the Headquarters Office of the Society is busier during the hot weather than at some other times.

A special activity of the summer months is the issuing of the yearly TRANSACTIONS. Although these volumes appear in the fall, they are all prepared during the summer. Indeed, this necessity was probably the original reason for omitting the two summer numbers of PROCEEDINGS. During the current season, yet another activity is going forward in the form of the compilation of an index to TRANSACTIONS, covering the years 1921–1934, inclusive. It is expected that this volume will be issued during the fall.

Standard Symbols and Abbreviations—An Ideal

Adopted for Use in Society Publications

IT IS at once characteristic and paradoxical that when a technologist undertakes to transmit his ideas for publication, he first writes his personal dictionary for that one paper. His intimate association with the subject of his specialty endows him with numerous preferences (personal prejudices) as to symbols and definitions of technical terms. It is thus that the practicing engineer who wishes to keep abreast of the progress in his field must, perforce, understand one or more lay languages and as many technical languages in addition as there are technologists to write them.

The American Standards Association is doing much to destroy this formidable "Tower of Babel" by establishing standard symbols and abbreviations for engineering use in many fields. It is sincerely to be hoped that writers on engineering subjects will more and more adopt the practice of making their work conform as nearly as possible to one of the standards in common use. With this aim in mind, lists of symbols in a number of prominent engineering fields are being printed here. All accord with the general practice of Society publications.

SYMBOLS FOR MECHANICS, STRUCTURAL ENGINEERING, AND TESTING MATERIALS

A list of standard symbols for mechanics, structural engineering, and testing materials was approved by the American Standards Association in January 1932 under the joint sponsorship of the American Society of Civil Engineers, the American Institute of Electrical Engineers, the American Association for the Advancement of Science, the Society for the Promotion of Engineering Education, and the American Society of Mechanical Engineers. These symbols are as follows:

Acceleration, angular	α (alpha)
Acceleration, due to gravity	g
Acceleration, linear	a
Angular distance	θ (theta)
Angular velocity	ω (omega)
Area	A
Axes, through any point	X-X, Y-Y, Z-Z
Breadth	b
Center of rotation	O
Coefficient of sliding friction	f
Concentrated load (same as force)	F
Constants	C
Curvature, radius of	ρ (rho)
Deflection	y
Deflection of a panel point of a truss	Δ (delta)
Density	ρ (rho) or d
Depth	d
Diameter	D
Distance, linear	s
Eccentricity of application of load	e
Efficiency (hydraulic, mechanical, volumetric)	e_h, e_m, e_v
Elasticity, modulus of	E
Elongation, unit	δ (delta)
Force	F
Force in any bar of a framed structure due to a load of unity applied at any point in any direction	u
Frequency (harmonic motion)	f or n
Gyration, radius of	k
Head	H or h
Height	h
Inertia, rectangular moment of	I
Inertia, polar moment of	J
Length	L
Load per unit distance	w
Load, total	W
Mass	m
Modulus of rupture	R
Moment in inch-pounds at any section of a girder due to the moment of one inch-pound applied to the girder at any point	m
Moment of force, including bending moment	M
Neutral axis, distance to extreme fiber	c
Number of revolutions per unit of time	n
Period (harmonic motion)	T
Power, horsepower	P
Pressure per unit of area	p
Radius	r
Ratio between modulus of elasticity of steel and modulus of elasticity of concrete	n
Ratio of the distance from the neutral axis to the outer fiber of a reinforced concrete beam to the distance from the outer fiber to the point of application of the resultant tensile stress	k
Ratio of the lever arm of the resisting couple in a reinforced concrete beam to the distance between the outer compressive fiber and the point of application of the resultant tensile stress	j
Reactions	R
Section modulus	Z or S
Statical moment of any area about a given axis	Q
Steel ratio, in reinforced concrete beams	p
Stress, unit	s
Stress, unit compressive	s_c
Stress, unit tensile	s_t
Stress, unit shear	s_s
Stress, total tensile or total steel, in reinforced concrete	T
Stress, total compressive or total concrete, in reinforced concrete	C
Stress, total shear	V
Stress, unit concrete, in reinforced concrete	f_c
Stress, unit steel, in reinforced concrete	f_s
Stress, unit shear of concrete	v
Temperature, absolute	T
Temperature, ordinary	t
Thickness	d or t
Time	t
Torque	T
Velocity, linear	V or v
Volume	V
Work, or energy	W

SYMBOLS FOR HYDRAULICS

The standard symbols for hydraulics, approved in July 1929 by the same sponsoring organizations, follow:

Acceleration:	
in general	a
due to gravity	g
Area	A
Channel flow:	
area of section	A
average velocity in section	V
depth of flow	d
hydraulic radius	R
hydraulic slope	S
length	L
surface width	B
Kutter's coefficient of roughness	n
Bazin's coefficient of roughness	m
Chezy's coefficient	C
Coefficient:	
of velocity	c_v
of contraction	c_c
of discharge	c_d
of roughness, Kutter's	n
of roughness, Bazin's	m
of Chezy	C

Density	ρ (rho)
Diameter	D
Energy per unit time (power)	P
Friction factor used in expressing pipe loss	f
Head:	
in general	h or H
elevation head	z
pressure head	h_p
velocity head	h_v
lost head	h, with appropriate subscript
Hydraulic radius	R
Hydraulic slope	S
Pressure:	
intensity of	p
total pressure (force)	F
Pipes:	
average velocity in section	V
diameter of	D
head lost in	h, with appropriate subscript
hydraulic radius	R
hydraulic slope	S
length	L
Power (energy per unit time)	P
Properties of water:	
density	ρ (rho)
bulk modulus of elasticity	K
Rate of discharge or flow (vol. per unit time)	Q
Slope, hydraulic	S
Time	t
Velocity:	
absolute	V
relative to moving casing	v
of moving casing	u
Viscosity:	
absolute	μ (mu)
kinematic	ν (nu)
(kinematic viscosity = absolute viscosity ÷ density)	
Weight:	
per unit volume	w
per unit time	W
Weirs:	
head as measured	H or h
velocity head of approach	h_0
crest height	Z
crest length	B
velocity of approach	v_0

Symbols Relating to Hydraulic Turbines and Pumps

Symbols Relating to Dimensions

Angle between the absolute velocity of the water and the velocity of the runner at any point, measured in degrees	α (alpha)
Angle between the relative velocity of the water and the velocity of the runner at any point, measured in degrees	β (beta)
Axial breadth or depth of runner entrance	B
Diameter of runner or impeller	D
Diameter of runner or impeller vanes at the middle of entrance space	D_1
Diameter of runner or impeller throat (inside diameter of band or shroud ring)	D_{th}
Radius to any point from center of runner or impeller	r

Symbols Relating to Efficiency

Hydraulic	e_h
Mechanical	e_m
Total or over-all	e

Symbols Relating to Head

Total head at any point	H
Potential head at any point	z
Pressure head at any point	h_p
Velocity head at any point	h_v

$$(H = h_v + h_p + z)$$

Symbols Relating to Power

Power, or energy per unit time	P
Power of turbine under 1-ft head	P_1
Power at brake	P_B
Power from water	P_W

NOTE: Where power is to be expressed in horsepower or other units, statement to that effect should be made.

Symbols Relating to Speed

Revolutions per minute	n
Revolutions per minute under 1-ft head	n_1
Specific speed or type characteristic	n_s

$$n_s = \frac{n\sqrt{P}}{H^{5/4}},$$

P being expressed in horsepower

Ratio of peripheral speed of runner to $\sqrt{2gH}$	φ (phi)

$$\varphi = \frac{u_1}{\sqrt{2gH}}$$

Symbols Relating to Velocity

Angular velocity, in radians per second	ω (omega)
Absolute velocity of the water at any point in a rotating runner or impeller	V
Circumferential velocity of a point on a rotating runner or impeller	u
Circumferential or tangential component of the absolute velocity of the water	V_u
Meridional component of the absolute velocity of the water (component in the plane containing the axis of rotation of runner or impeller)	V_m
Radial component of the absolute velocity of the water	V_r
Relative velocity of the water with respect to the moving runner or impeller	v

NOTES: (1) Subscripts 1 and 2 may be used to refer to the points of entrance and discharge in either a runner or impeller, the assumption being that the water always flows from point (1) to point (2). Thus V_1 and V_2 refer to the absolute velocity of the water at entrance and exit from a runner or impeller. (2) The German standard symbols for absolute, relative and circumferential velocities are c, w and u; and γ (gamma) is used for unit weight. The large amount of German literature on the subject of turbines warrants the statement here of these facts.

Standard Abbreviations

The following general rules for abbreviations have been adopted by the Society in collaboration with the other Founder Societies. Distinction should be made between symbols for abstract quantities and the abbreviation for the concrete units. For example, in the formula,

$$Q = AV$$

the symbol Q stands for the rate of discharge, A for area of section, and V for average velocity; whereas the units in which Q is measured may be shown by the abbreviations "cu ft per sec," those in which A is measured by "sq ft," and those in which V is measured by "ft per sec."

Terms denoting units of measurement should be abbreviated in the text only when preceded by the amounts indicated in numerals; thus "several inches," "1 in.," "12 in." In tabular matter, specifications, maps, drawings, and tests for special purposes, the use of abbreviations should be governed only by the desirability of conserving space.

A sentence should not begin with a numeral followed by an abbreviation.

Short words such as "ton," "day," and "mile" should be spelled out.

Abbreviations should not be used where the meaning will not be clear. In case of doubt, spell out.

The use of conventional signs for abbreviations in text is not recommended; thus "per," not /; "lb," not #; "in.," not ″. Such signs may be used sparingly in tables and similar places for conserving space.

In the interest of economy and the reduction of waste, the elimination of the period is recommended except where such an omission results in an English word. This exception is not followed in the case of a few mathematical and chemical terms, such as sin, tan, log, As, etc.

The use in text of exponents for the abbreviations of "square" and "cube" and of the negative exponents for terms involving "per" is not recommended. The superior figures are usually not available on the keyboards of typesetting and linotype machines and composition is therefore delayed. These shorter forms are permissible in tables and are sometimes difficult to avoid in text.

Forms in general use as abbreviations follow:

Acre	acre
Acre-foot	acre-ft
Air horsepower	air hp
Average	avg
Barrel	bbl
Board feet (feet board measure)	fbm
Brinell hardness number	Bhn
British thermal unit	Btu
Bushel	bu
Kilometer	km
Kilometer per second	km per sec
Kilovolt	kv
Kilowatt	kw
Latitude	lat
Linear foot	lin ft
Liter	liter
Longitude	long.
Maximum	max
Meter	m
Miles per hour	miles per hr
Centigram	cg
Centimeter	cm
Centimeter-gram-second (system)	cgs
Circular mils	cir mils
Cubic	cu
Cubic centimeter	cu cm
Cubic foot	cu ft
Cubic feet per second	cu ft per sec
Cubic inch	cu in.
Cubic meter	cu m
Cubic millimeter	cu mm
Cubic yard	cu yd
Degree	deg or °
Degree Centigrade	C
Degree Fahrenheit	F
Dozen	doz
Feet per minute	ft per min
Feet per second	ft per sec
Foot	ft
Foot-pound	ft-lb
Foot-pound-second (system)	fps
Gallon	gal
Gallons per minute	gal per min
Gallons per second	gal per sec
Gram	g
Hectare	hectare
Horsepower	hp
Horsepower-hour	hp-hr
Hour	hr
Hundredweight (112 lb)	cwt
Inch	in.
Inch-pound	in-lb
Inches per second	in per sec
Kilogram	kg
Kilogram-meter	kg-m
Kilograms per cubic meter	kg per cu m
Kilograms per second	kg per sec
Miles per hour per second	miles per hr per sec
Millimeter	mm
Million gallons per day	mgd
Minimum	min
Minute	min
Minute (angular measure)	′
Ounce	oz
Ounce-foot	oz-ft
Ounce-inch	oz-in.
Parts per million	ppm
Pint	pt
Pound	lb
Pound-feet	lb-ft
Pound-inch	lb-in.
Pounds per square foot	lb per sq ft
Pounds per square inch	lb per sq in.
Quart	qt
Revolutions per minute	rpm
Revolutions per second	rps
Second	sec
Second (angular measure)	″
Second-feet	cu ft per sec
Square	sq
Square centimeter	sq cm
Square feet	sq ft
Square inch	sq in.
Square kilometer	sq km
Square meter	sq m
Square millimeter	sq mm
Ton	spell out
Ton-mile	spell out
Yard	yd
Year	yr

American Engineering Council

ENGINEERING AFFAIRS AT WASHINGTON

NATIONAL projects of interest to engineers continue to follow each other with bewildering rapidity at Washington. During the last month engineers have been related to at least one field with which they have not been officially associated, namely, the Home Owners' Loan Corporation. This organization administers the distribution of some two hundred millions of dollars to be used for modernizing and reconditioning homes. A group of 12 regional supervisors has been established, each in charge of men with a background of architectural or engineering experience. Under these regional supervisors, state organizations have been established for considering the equities involved in the applications for this remodeling development.

It is quite probable that the broad program of housing, the details of which have not yet been made public, will also bring opportunities for engineers and engineering in the follow-through of the plans. This particular operation is under the general direction of Harry L. Hopkins, Federal Emergency Relief Administrator.

MEETING OF EXECUTIVE COMMITTEE OF AMERICAN ENGINEERING COUNCIL

For June 22, President Coleman has called a meeting of the Executive Committee of the American Engineering Council to discuss several important phases of national policy and their relations to the Council, and also several matters dealing with the development of the Council organization itself, especially the policy having to do with such subjects as development of our water resources, the relation of civil service to engineers' development, the question of compensation of engineers, and recommendations from the committees on aeronautics, communications and other matters of public interest. The committee will also discuss the admission of new members, who have made application to the Council, and plans for a wider representation of local and state engineering groups in the Council.

REFERENDUM ON PUBLIC WORKS EXPENDITURES

During the past month, at the request of President Coleman, and as a result of a mail vote by the Executive Committee, two questions were submitted to the Council with regard to its policy on public works. The questions are as follows:

1. Do you approve of the Council's actively supporting in any legitimate way the various proposals to extend credit

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for the development of private construction enterprise based upon the requirement of adequate security?

31—yes
2—no

2. Shall the Council support a movement to obtain additional PWA funds?

20—yes
13—no

Each question is recorded with the votes by the Council members for or against the proposals.

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News of Local Sections

Chattanooga Section

A meeting of the Chattanooga Section was held on April 5. Various business matters were discussed, as was the question of the advisability of employing engineers on the new Program of Public Works, which replaces the CWA. A committee was employed to make a thorough study of the situation and to take up with local authorities the matter of employing competent engineers.

Cleveland Section

Members of the Student Chapters of the Case School of Applied Science and Akron University were present at a meeting of the Cleveland Section held on May 8 at the Case Club. There was an attendance of 72 at both the dinner and the meeting. The first speaker on the program was Robert Hoffmann, consulting engineer on public works for the City of Cleveland. He was followed by Dr. Wickenden, president of the Case School, who advised recent graduates as to the best method of finding work of an engineering nature. The meeting was then turned over to the Case School of Applied Science Student Chapter, and five students gave brief outlines of their special theses, followed by laboratory demonstrations of the problems being considered. On June 5 a luncheon meeting of the Cleveland Section was held, with 31 present. Following a business session, J. E. Hyde, of Western Reserve University, gave an illustrated talk on the topic, "Cleveland Geology for Engineers."

Georgia Section

A luncheon meeting of the Georgia Section was held on May 7. There were 55 present at this session, which was largely given over to a discussion of timely business matters. A stimulating talk by N. W. Dougherty, Professor of Civil Engineering at the University of Tennessee, was the final feature of the program. Professor Dougherty outlined the history of engineering, emphasizing particularly the fact that, although the profession goes back several thousand years, its real development has taken place in the past century.

Illinois Section

The Illinois Section recently held three interesting meetings. On April 27 Robert Kingery, Director of Public Works for the State of Illinois, addressed a session. On May 11 Frank A. Randall, structural engineer of Chicago, and J. C. Sanderson, construction engineer for Sargent and Lundy, of Chicago, gave informal talks on subjects of current economic and engineering interest. And the meeting held on June 1 was the occasion of making the annual prize awards to certain graduates of engineering schools and colleges.

Kansas City Section

The Kansas City Section held a dinner meeting at the University Club on May 25. Among the speakers were R. E. McDonnell, of the Burns and McDonnell Engineering Company, who spoke on the subject of Muscle Shoals, and R. W. Knight, District Traffic Manager of the United Air Lines, who discussed air transportation. An interesting motion picture film was shown in connection with the latter talk.

Los Angeles Section

On May 9 the Los Angeles Section met at the California Institute of Technology. Before dinner was served the members had an opportunity to examine the operation of a large-scale model of the tidal and current movements in Alamitos Bay at the mouth of the San Gabriel River. Dinner was followed by a routine business session and the presentation of two illuminating addresses. These were given by Dr. Robert Andrews Millikan, who spoke on "The Relation of the National Science Advisory Board to Engineering," and Irving P. Krick, instructor in meteorology at the California Institute of Technology, who chose for his topic, "New Developments in Meteorology Pertaining to Storms and Precipitation." There were 223 present at the dinner, and 270 at the meeting.

Maryland Section

The Maryland Section met at the Engineers' Club in Baltimore on April 4, with Harrison P. Eddy, President of the Society, as guest of honor. Mr. Eddy spoke briefly of the work being done by the Society in the preparation of the engineering section of the NRA construction code. Then Col. Donald H. Sawyer, director of the Federal Employment Stabilization Board, of the U. S. Department of Commerce, discussed the subject, "National, Regional, and State Planning." In his talk Colonel Sawyer stressed the difficulties that the Government has encountered in its relief work as the result of a lack of planning and pointed out the increasing need of such planning in the future.

Milwaukee Section

A meeting of the Milwaukee Section was held at the City Club on March 22. The feature of the occasion was a talk by James Ferebee, State Director of the Public Works Administration, who discussed PWA work in its relation to the local sewage disposal plant. At a meeting of the Section held on April 26 the speaker was R. C. Johnson, a member of the examining board for architects and civil engineers, who stressed the value of national and state planning in the improvement of present economic conditions. An animated discussion from the floor followed his address.

Philadelphia Section

An inspection trip to the estate of Pierre S. du Pont, near Kennett Square, Pa., and to Wilmington, Del., was enjoyed by members and guests of the Philadelphia Section on May 16. In Wilmington the party inspected the new dam and other features of the public water supply as well as the marine terminal. In the evening the group returned to Philadelphia for dinner and a meeting at the Engineers Club. The speaker of the occasion was Charles H. Stevens, chief engineer of the Philadelphia Department of City Transit, who emphasized the value of membership in the Society to young engineers. The attendance at the dinner was 41, and at the meeting 49.

Texas Section

The spring meeting of the Texas Section was held at the Plaza Hotel in San Antonio on May 11 and 12. The first day was devoted largely to the presentation of a technical program. Among those contributing interesting papers at this session were the following: Willard Simpson, president of the W. E. Simpson Company, Inc., of San Antonio, who dealt with the subject, "Foundation Experiences in Texas"; O. N. Floyd, consulting engineer of Dallas, whose paper described the Red River basin project; Homer Stevenson, who treated the topic, "The Design and Construction of an Earthquake-Proof Dam in Mexico"; and W. S. Stanley, chief chemist of the San Antonio sewage treatment plant, who gave an illustrated discussion of the operation of this plant. These topics elicited much enthusiastic discussion. The morning of May 12 was given over chiefly to business, and in the afternoon a picnic and barbecue were enjoyed at Medina Lake. The total registration for the meeting was 107.

ITEMS OF INTEREST

Engineering Events in Brief

Civil Engineering for August

By a U. S. Supreme Court decision made in 1931, the Metropolitan District Water Supply Commission of Boston is permitted to divert and use all the flow of the Ware River above 132 cu ft per sec (85 mgd). As designed and constructed, the diversion works are capable of handling as much as two billion gallons a day entirely automatically. The diversion is effected by dropping the water down a spirally lined shaft 260 ft deep, into the 25-mile tunnel connecting the existing Wachusett Reservoir with the Quabbin Reservoir, now under construction. The flood waters of the Ware River are skimmed off the flow, and the remainder is allowed to continue down the river. The energy in the water falling in the shaft is absorbed by a cast-iron lining, which is fitted with helical vanes, imparting a rotating motion to the water, which is held against the lining by centrifugal force. The design and the interrelation of the automatic devices will be explained in an article prepared for the August issue by the designer, Karl R. Kennison, M. Am. Soc. C.E.

Much study has been given to the design and efficiency of highway guard rails by Searcy B. Slack, M. Am. Soc. C.E., as described in another article for the August issue. An automobile out of control on a highway must be kept from running off bridge approaches, high fills, and banks. Although a stone wall might successfully stop the car, the resulting impact would be likely to do more damage to the vehicle and its occupants than would running off the road. More resilient types of rails are therefore required. From a study of various available types of guard rails and laboratory and field tests on full-size specimens in Georgia, Mr. Slack makes helpful observations concerning the use of some available types of guard rails.

By an act of Congress in 1927, the Corps of Engineers was directed to report on the more important navigable streams of the United States, and to make an analysis of the needs and possibilities for the improvement of navigation and the development of hydro-electric power for the control of floods and for irrigation. One of these reports, that on the Delaware River, is of importance because New York City, Philadelphia, and cities in New Jersey and the adjacent industrial areas, including a population of $13^1/_2$ million people, will probably find it necessary to draw on this river in future for both water supply and power. A résumé of this report, now in the hands of the Government printer, has been made by Mason J. Young, M. Am. Soc. C.E., Major, Corps of Engineers, and will appear in the August issue.

Another article in preparation for August is by E. W. Seeman, Assoc. M. Am. Soc. C.E., Engineer for the Merritt-Chapman and Whitney Corporation, who describes the application of a recently developed method of concrete placing to massive construction. In order to provide a 458-mile reach of the Upper Mississippi with a 9-ft navigation channel, a series of roller-gate dams and locks is required to form the pools. At Dam No. 5, about a hundred miles below St. Paul, some unusual methods are being used for constructing the concrete slabs and piers. Here the Merritt-Chapman and Whitney Corporation is pumping unbelievably dry concrete through 7-in. pipe lines to points as far as 1,000 ft from the mixing plant. Operations were continued during the past winter in temperatures as low as —10 F.

Sizes of Paving Brick Standardized

Simplified practice recommendation R1-32, covering vitrified paving brick, has been reaffirmed, without change, as of June 15, 1934, by the Standing Committee of the industry, in charge of the periodic review of this program. This recommendation, which was first proposed and developed by the industry in 1921, has since that time been reviewed annually and revised seven times. In its recent survey, the committee found that 91.5 per cent of the total shipments during the calendar year 1933 were in accordance with the six sizes shown in the Simplified Practice Recommendation.

In 1921 vitrified paving brick was available in 66 different sizes. The original recommendation reduced this variety to 11, and the first revision conference further reduced it to 7. Subsequent conferences and surveys have resulted in a net reduction to 6 sizes.

This waste-elimination program for the paving brick industry is a part of the work of the Division of Simplified Practice of the U. S. Department of Commerce. Copies of the recommendation covering this industry may be secured from the Superintendent of Documents, Government Printing Office, Washington, D.C., for 5 cents each.

South Carolina Proclaims Engineers' Day

By proclamation, Governor I. C. Blackwood of South Carolina set aside June 15, just past, as Engineers' Day, to do honor to the engineers of the state. The Governor recommended that the state and county departments send their engineers to the summer convention of the engineers of the state, which was held at Columbia, S. C., on June 14 and 15.

In his proclamation the Governor said:

"It seems well to honor a group of citizens who though specially generous in their contributions to our progress and comfort have seldom received public commendation. . . . South Carolina is grateful to her engineers. Public and private enterprise has entrusted hundreds of millions to engineers. . . . They have builded well, not ostentatious monuments to inspire awe, as did the ancients, but useful wonders which lighten the housewife's and the laborer's burdens, speed the message, improve our goods, and make life more pleasant."

The convention at Camp Jackson, Columbia, was held under the auspices of the South Carolina Society of Engineers and was participated in and attended by many members of the Society.

Engineers Awarded Degrees

Academic recognition of the engineer's contribution to science makes itself particularly evident at commencement time when honorary degrees are bestowed on those who have made outstanding contributions to the profession. Among the engineers thus honored during the 1934 commencement season are several members of the Society. In addition to the following list, there are doubtless others regarding whom information has not yet been received at Society Headquarters.

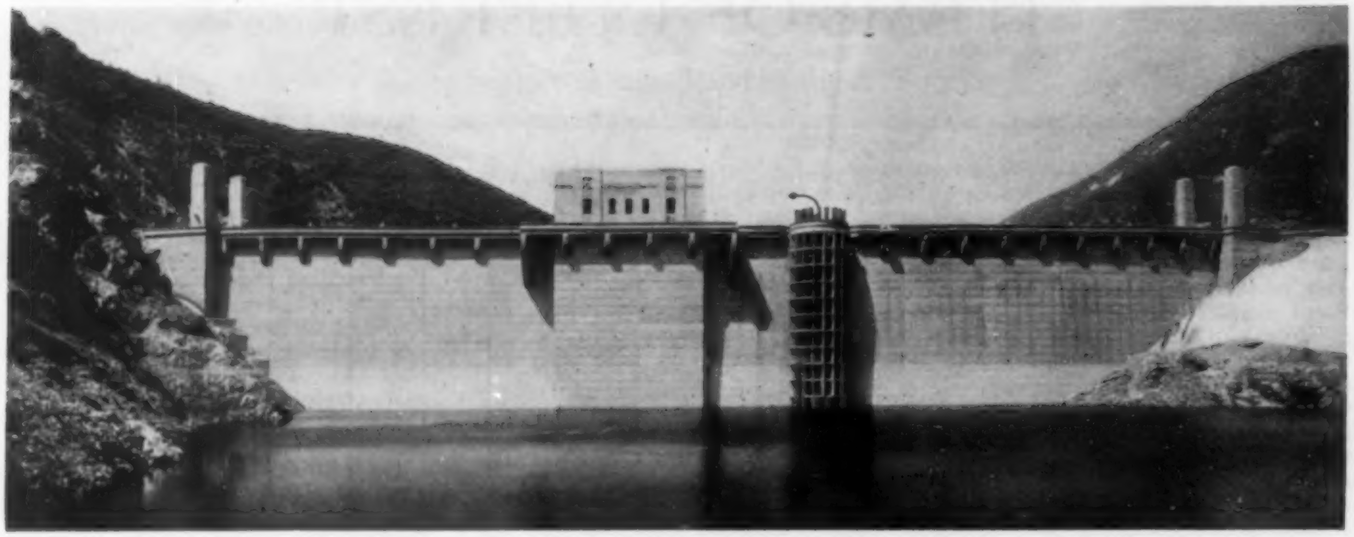
Herbert S. Crocker, Past-President Am. Soc. C.E., Doctor of Science, University of Colorado, and Doctor of Engineering, University of Michigan.

C. B. McCullough, M. Am. Soc. C.E., Doctor of Engineering, Oregon State College.

Elwood Mead, M. Am. Soc. C.E., Doctor of Laws, University of Wyoming.

J. L. Savage, M. Am. Soc. C.E., Doctor of Science, University of Wisconsin.

SIXTY-FOURTH ANNUAL CONVENTION of the Society, July 11–14, 1934, in Vancouver, British Columbia

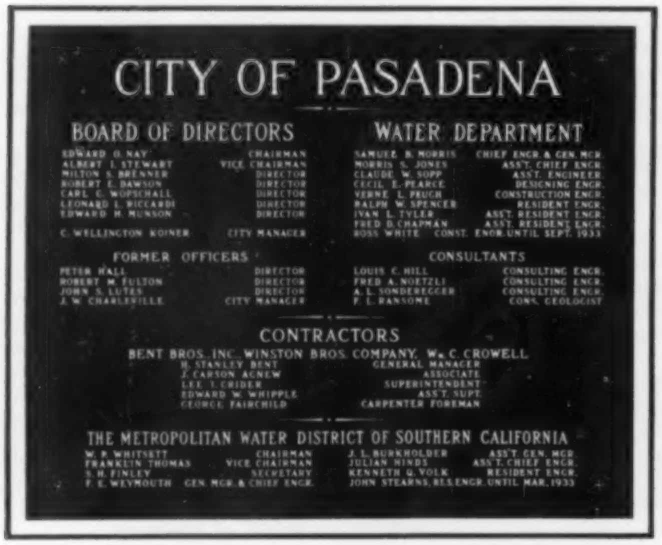


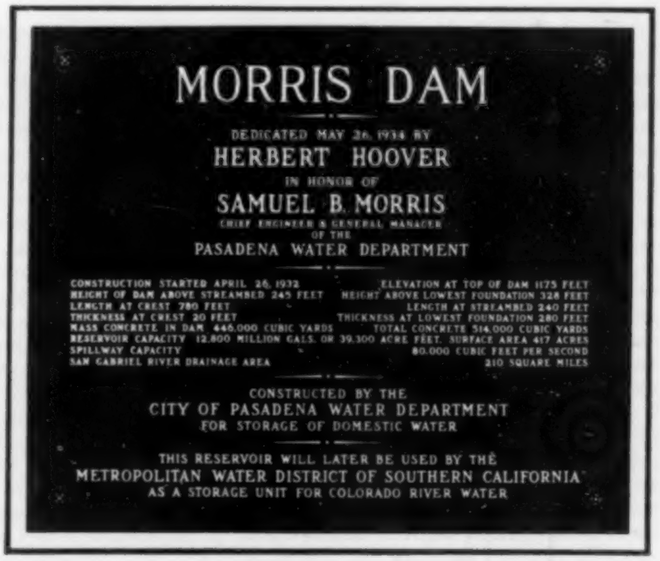
Engineers Recognized in Dedication of Morris Dam

FOR NEARLY ten years the City of Pasadena, Calif., has been struggling to finance and construct a dam across the San Gabriel River to store its flood waters and add to the domestic supply of the city. When, on May 26 of this year, the dam was dedicated by Herbert Hoover, Hon. M. Am. Soc. C.E., former President of the United States, and the valve in the conduit to Pasadena was opened by Samuel B. Morris, M. Am. Soc. C.E., the chief engineer, the hopes of the city were realized.

Dams have been named for presidents of the United States, for sponsors, and for localities. Less often have they been named for engineers, and even less frequently for those who had charge of their design and construction. Morris Dam is a notable exception. Not only has it received the name of its chief engineer but the dedicatory plaques bear as well the names of all the engineers connected with its design and construction, many of whom, it will be noted, are members of the Society. The plaques also contain the names of the contractors and of the city officials under whose direction the dam was built.

Pasadena is a member city of the Metropolitan Water District of Southern California, which is now engaged in constructing an aqueduct to bring the waters of the Colorado River to the cities of Southern California. At some future time, coordinated with the completion of this aqueduct, the reservoir formed by Morris Dam will provide terminal storage for water from this river. Thus this structure is part of Pasadena's contribution to the completed Colorado River Aqueduct Project.





BUILDERS OF MORRIS DAM HONORED BY DEDICATORY PLAQUES

Engineering—Profession or Vocation?

By J. JAMES KNOX, JUN. AM. SOC. C.E.

ELMHURST, N.Y.

Presented at the meeting of the Metropolitan Section on May 16, 1934, and awarded first prize in the Effective Speaking Contest of the Junior Branch of this Section.

IN AN ADDRESS delivered at the inaugural meeting of the Columbia University Student Chapter of this Society, the late William Barclay Parsons, Hon. M. Am. Soc. C.E., remarked: "So wide has become the field of engineering and so varied its applications that there is room within its folds for both a vocation and a profession." The statement on its face may seem self-evident, yet how many engineers realize the implications involved?

Shall the individual engineer determine for himself whether he is practicing a profession or following a vocation; whether he is to consider himself bound by the obligations laid upon professional men or possessed of the freedom of action accorded to those engaged in purely commercial pursuits? There is not, for ex-

ample, anything inherently wrong in the attempt on the part of one engineer to compete, directly and openly, with another; to urge his own superior qualifications while pointing out his competitor's weaknesses; and to offer his services at whatever "cut rate" he chooses. That is traditional and quite respectable commercial practice. But it is not "professional."

That brings us to the question of what is really meant by the term "profession." From the standpoint of society it must be an occupation of such importance, and involving work of such a responsible nature, that only men of high ability and character can be permitted to enter it. And society has the right to demand from its professional men that their abilities and energies shall be devoted primarily to the practice of their profession, which in turn implies a measure of responsibility on the part of the community towards its professional men.

Of more direct interest to us is the conception of what the term profession means to the individual engineer, or more particularly to the young man just hoping to enter upon his career. From his entrance into engineering school, it has been impressed upon him that he has not chosen an occupation in which he may expect to become wealthy. He knows and accepts the fact that he will have to work all his life, and that the work will be responsible and arduous. But he has chosen engineering because it appeals to him as being a field worthy of every bit of energy and ability which he can bring to it. There need be no cant about the "joy of devoting one's life to the service of mankind." There is merely a frank recognition on the part of the young engineer that in the doing of work which he enjoys and which he himself thinks worth while a man finds his own greatest satisfaction in life.

The cynical remark has been made that today "honesty is a luxury which few can afford"—and there is grim truth in the statement. We might paraphrase it and say that the ideals and standards of professional men are today luxuries which but few engineers can afford.

For we face continually the inescapable fact that each one of us must have an income, whether it be in the form of pay for services rendered or some sort of sugar-coated charity, and in the case of a young man it must be an increasing income. Let us not by hypocritical about this. A young man whose income is not increasing steadily is failing. He has no right to marry, to have children, to assume those normal responsibilities which make life worth living to the individual, unless he can hold his own financially. And the doing of that is a problem which each young man must face for himself.

If as a professional man—that is, one whose talents and energies are devoted primarily to the practice of his profession—he cannot command a living income, then he must turn his chief attention to the problem of money making pure and simple, regarding the practice of his profession as being merely a means to that end. But with that attitude, can he be said to be "practicing a profession?"

Irrespective of individual problems, the community must have professional engineers upon whose disinterested service it can rely; men who are not concerned first and foremost with their own and their families' existence. For the protection of society and in justice to the individual there should be a far more precise distinction between the professional and the vocational phases of engineering than exists at present. No one spectacular act can accomplish this; there must be continuous, concerted action on the part of universities, licensing boards, and the professional societies, tending strictly to define the functions, qualifications, and obligations of the professional engineer, thus at the same time implicitly defining the non-professional or vocational worker. These latter could then feel free to advance their own best interests as they might see fit, through unions if that seemed advisable, neither claiming the privileges nor bound by the obligations of a profession.

Equality of opportunity to meet increased standards must of course be preserved. But that does not mean that every young man who might like to be an engineer has the right to be admitted to the profession. He has the right only to every opportunity to demonstrate his ability, and to be judged on that basis.

The medical profession, for both the public's protection and its own, has been forced to curtail sharply the number of men it admits, and in general this is done justly. But done it had to be. Society could not tolerate the spectacle of a dozen surgeons competing for the privilege of amputating a leg or removing an appendix, no matter how much the individual surgeons might need the fee. No more can it tolerate the obvious dangers involved when engineers attempt to underbid each other for the opportunity of designing a bridge or a dam.

Thus we have the problem: on the one hand society, demanding that protection which is its right; on the other the individual, demanding the opportunity to make a living, by ruthless, unsparing competition if driven to it, in what is still essentially a competitive economic order.

The standards of a profession cannot be maintained by giving lip service to lofty generalities, but only by strict regulation and definite sanctions. And if the task be neglected? Then in the future when we refer to the profession of engineering we may be greeted with that same ugly, bitter laugh that today meets our money-changers when they refer to the "profession" of banking.

NEWS OF ENGINEERS

From Correspondence and Society Files

Joseph B. Paulson, Jr., is now materials inspector in the Engineering Office of the U. S. Army in St. Paul, Minn.

Harold Miller Pearson, formerly an inspector for the Alameda County (California) Civil Works Administration, has recently been appointed to a position as junior engineer with the State of California Division of Highways.

Maurice H. Belkin has resigned his position as civil engineer and architect for Irving Feldman, Inc., of New York, N.Y., to accept a connection in a similar capacity with the MacArthur Construction Company, of the same city.

Fred L. Moore, formerly associated with Chase and Waring, of New York, N.Y., has recently been appointed inspector of buildings for Garden City, N.Y.

William T. Haight has accepted a position as Assistant Bridge Construction Engineer of the San Francisco-Oakland Bay Bridge, with offices in San Francisco, Calif. He was formerly with Frederic W. Teschke, of Hollywood, Calif.

Carl A. Bock, formerly vice-president of the Dayton Morgan Engineering Company, of Dayton, Ohio, is now assistant chief engineer for the Tennessee Valley Authority, with headquarters in Knoxville, Tenn.

Charles C. Cragin is now with the Western Gas Company, of El Paso, Tex. He was previously general manager and chief engineer of the Salt River Valley Water Users Association, of Phoenix, Ariz.

Merrill C. Lorenz has resigned as construction foreman with the National Park Service on Emergency Conservation Work to accept a position as inspector of general construction on Lock and Dam 18 on the Mississippi River, at Burlington, Iowa.

Melvin L. Enger, head of the Department of Theoretical and Applied Mechanics of the University of Illinois since 1926, has recently been elected dean of the College of Engineering and director of the Engineering Experiment Station of the university.

Oliver A. Johnson is now employed as an inspector at the U. S. Waterways Experiment Station, in Vicksburg, Miss.

F. W. Hanna has resigned as chief engineer and general manager of the East Bay Municipal District, with headquarters in Oakland, Calif., to devote his time to the practice of consulting engineering and to writing. He is at present in Ankeny, Iowa.

Robert F. Cowles, formerly with the Arrowhead Lake Corporation, of Arrowhead Lake, Calif., is now employed as supervising engineer by the U. S. Coast and Geodetic Survey for work on Kennett Reservoir. His headquarters are at Kennett, Calif.

Fred M. Berry has severed his engineering connection with the Stone and Webster Engineering Corporation, of Wenatchee, Wash., to become Chief of Party on Grand Coulee Dam, with headquarters in Almira, Wash.

F. J. GASTON, consulting civil engineer of Havana, Cuba, has recently been made professor of technical English in the School of Engineers and Architects of the University of Havana.

JOHN S. LANE has accepted a position as inspector for the Los Angeles County Flood Control District, with offices in Los Angeles, Calif.

JOHN N. PIROK is now Engineer Technician for the State of Illinois Emergency Conservation Work of the Civilian Conservation Corps, with headquarters in Staunton, Ill.

T. E. VELTFORT has severed his connection with Stone and Webster, Inc., of New York, N.Y., to become assistant secretary of the Copper and Brass Mill Products Association, of the same city.

ARTHUR L. ELLIOTT is now Junior Bridge Construction Engineer with the State of California Highway Bridge Department on the construction of the San Francisco–Oakland Bay Bridge.

CHESTER MUELLER, formerly Principal Assistant Engineer of the New Jersey Department of Public Affairs, has established a law office at 850 Broad Street, Newark, N.J.

Changes in Membership Grades

Additions, Transfers, Reinstatements, Deaths, and Resignations

From May 10 to June 9, 1934, Inclusive

Additions to Membership

AURIEMMA, ALFRED ANTHONY (Jun. '33), 245 Danforth Ave., Jersey City, N.J.

BAYOT, JEAN MARIE (Assoc. M. '34), Apartado 663, Caracas, Venezuela.

BREEDING, SETH DARNADY (Assoc. M. '34), Asst. Engr., U. S. Geological Survey, Box 267, Austin, Tex.

CAWLEY, CLIFFORD COMER (Jun. '34), Senior Draftsman, Design Office, Los Angeles County Flood Control Dist., 1234 West 50th St., Los Angeles, Calif.

CLOUDMAN, CHARLES GREENLEAF (Jun. '33), St. George Hotel, Brooklyn, N.Y.

DER, JACK HIM (Jun. '34), Asst. Engr., Chekiang Highway Administration, Hangchow, China.

DETRICK, DANA FARRINGTON (Jun. '34), 1027 Waverly St., Palo Alto, Calif.

DEWELL, ROBERT DIEVENDORF (Jun. '34), Draftsman, U. S. Coast and Geodetic Survey; 2110 Santa Clara Ave., Alameda, Calif.

FORBESS, ORDIS ELDON (Jun. '34), Rodman, State Highway Dept., Box 442, Tulia, Tex.

FRANKLIN, GEORGE EDWARD (Assoc. M. '34), Supt., Streets and Bridges, City of Dallas (Res., 5027 Mission St.), Dallas, Tex.

GIBSON, COUNT DILLON (M. '34), Prof. and Head, Dept. of Geology, Georgia School of Technology, Atlanta, Ga.

GRAHAM, NATHAN JEROME (Jun. '34), Upper Lake, Calif.

HOFMANN, OLIVER DIMMITT (Jun. '34), Timekeeper, Dimmitt & Taylor, Los Angeles (Res., 4851 La Roda, Eagle Rock), Calif.

HOLMES, JOSEPH MARK (Jun. '34), Junior Topographic Engr., U. S. Geological Survey, Washington, D.C.

HUBBARD, JAMES WILLIAM (Assoc. M. '34), Supt. and Gen. Mgr., The Smethport Water Co., Smethport, Pa.

JAROS, STANLEY FRANCIS (Jun. '34), Chagrin Falls, Ohio.

KAISER, EDGAR FOSBURGH (Jun. '34), 1522 Latham Sq. Bldg., Oakland, Calif.

KINGMAN, IRVING HALL (Jun. '34), 164 Percy St., Flushing, N.Y.

KISSAM, PHILIP (Assoc. M. '34), Asst. Prof., Civ. Eng., Princeton Univ. (Res., 15 Newlin Rd.), Princeton, N.J.

LAWTON, ELMORE GRENVILLE (Jun. '33), Care Thacker Coal & Coke Co., Thacker, W.Va.

LIGGETT, PHILIP TAZWELL (Jun. '33), 1023 Bates Ave., Kansas City, Mo.

McCONNELL, JOHN WALDO (Assoc. M. '34), Field Engr., Westchester County Park Comm.; Seneca St., Dobbs Ferry, N.Y.

MAY, DAVID CHAPIN (Assoc. M. '34), Asst. Engr., Met. Water Dist. of Southern California (Res., 9 Roosevelt Rd.), Banning, Calif.

MILES, THOMAS KIRK (Jun. '34), Asst. Supt., Hawaiian Dredging Co., Ltd., Eleele, Kauai, Hawaii.

MODAK, BHASKAR LAXMAN (M. '34), Chf. Engr., Holkar State, Indore, Central India.

PEARSON, EDWARD RUSSELL (Jun. '34), Junior Engr., U. S. Engr. Office, Box 668, Baton Rouge, La.

ROBERTS, DWIGHT FULTON (Assoc. M. '34), With State Highway Comm., Bridge Constr. Dept.; 225 Roosevelt St., Fullerton, Calif.

RUDDER, IRVING AVRUM (Jun. '34), 287 Linden Boulevard, Brooklyn, N.Y.

STAUB, WILLIAM SHAFFER (Jun. '34), Mgr., The East Rainelle Water & Plumbing Co., East Rainelle, W.Va.

SUTPHIN, FRANCIS STOWERS (Assoc. M. '34), Senior Design Draftsman, Los Angeles County Flood Control Dist., Los Angeles (Res., 1440 Lorain Rd., San Marino), Calif.

TINGEY, WILLIS ALMA (Jun. '34), Junior Engr., U. S. Geological Survey, Washington, D.C.

VAN ORMAN, CLARE RALSTON (Jun. '34), Junior Engr., U. S. Engr. Office, Missouri River Div., Hydr. Section (Res., 1915 Central Ave.), Kansas City, Kans.

Membership Transfers

BIEMANN, BERNHARD FREDERICK (Jun. '22; Assoc. M. '34), With Home Title Guaranty Co., 51 Willoughby St., Brooklyn (Res., 109–10 Park Lane South, Richmond Hill), N.Y.

CLUTE, HAROLD MOORE (Jun. '28; Assoc. M. '34), Junior Engr., Eng. Service Div., Tennessee Val. Authority, Nitrate Plant, Ala.

COWIE, GEORGE DURNO (Assoc. M. '23; M. '34), Hydr. and Geodetic Engr., Officer in Chg., New York Field Station, U. S. Coast and Geodetic Survey, Room 741, Custom House, New York, N.Y.

DUNHAM, CLARENCE WHITING (Assoc. M. '27; M. '34), Asst. Engr., Design Div., Port of New York Authority, 111 Eighth Ave., New York, N.Y. (Res., 23 Prospect Terrace, East Orange, N.J.)

HARRJE, HENRY JOHN (Jun. '31; Assoc. M. '34), Bldg. Supt., Oppenheim, Collins & Co., 534 Main St. (Res., 655 La Salle Ave.), Buffalo, N.Y.

HOOVEL, WILLIAM HENRY (Jun. '27; Assoc. M. '34), Asst. Engr., Bureau of Eng., City of Waterbury (Res., 42 Waterville St.), Waterbury, Conn.

TOTAL MEMBERSHIP AS OF JUNE 9, 1934

Members	5,752
Associate Members	6,284
Corporate Members	12,036
Honorary Members	17
Juniors	3,162
Affiliates	107
Fellows	4
Total	15,326

KEAST, SCHUYLER SHELDON ALBERT (Assoc. M. '21; M. '34), Asst. Engr., Dept. of City Transit City Hall Annex (Res., 4845 North 18th St.), Philadelphia, Pa.

KELKER, JAMES JOSEPH ARTHUR (Jun. '30; Assoc. M. '33), Civ. Engr., The Mid-Kansas Oil & Gas Co., Box 1129, Shreveport, La.

KRAMER, HANS (Jun. '19; Assoc. M. '24; M. 34), Capt., Corps of Engrs., U.S.A. Asst. to Dist. Engr., U. S. Engr. Office, Box 443, Memphis, Tenn.

LEWIS, DUDLEY LELAND (Assoc. M. '23; M. '34), City Engr., City Hall, Fort Worth, Tex.

McATEE, FRAYNE LEIGH (Jun. '31; Assoc. M. '34), With Bureau of Highways, State Dept. of Public Works (Res., 500 Franklin St., Apartment 6), Boise, Idaho.

PETERSON, EDWARD JOHN LAWRENCE (Jun. '26; Assoc. M. '34), Asst. Maintenance Engr., Div. of Highways, Dist. III (Res., 716 H St.), Marysville, Calif.

RICHARDSON, GEORGE SHERWOOD (Assoc. M. '25; M. '33), Bridge Design Engr., Dept. of Planning, Allegheny County, County Office Bldg. (Res., 310 South Home Ave., Bellevue), Pittsburgh, Pa.

SOUCEK, JAROMIR JAN (Jun. '29; Assoc. M. '34), Cons. Engr., Vystavni 24, Brno, Czechoslovakia.

TIMBY, ELMER KNOWLES (Jun. '29; Assoc. M. '34), Instr., Eng., School of Eng., Princeton Univ. (Res., 68 Wiggins St.), Princeton, N.J.

Reinstatements

JOHNSON, ANDREW KINGHORN, M., reinstated May 9, 1934.

Resignations

EDWARDS, RAY OMER, Assoc. M., resigned Feb. 15, 1934.

McNEE, THOMAS LEE, Jun., resigned May 11, 1934.

SAGE, WILLIAM HAMPDEN, JR., M., resigned May 29, 1934.

ZARTARIAN, LEVON, Jun., resigned June 4, 1934.

Deaths

CORNISH, LORENZO DANA. Elected Jun., April 5, 1904; Assoc. M., Feb. 7, 1906; M., June 30, 1910; died May 12, 1934.

CURTIS, BENJAMIN JOHN. Elected Jun., Sept. 2, 1914; Assoc. M., June 16, 1919; M., Jan. 17, 1927; died May 19, 1934.

EVQUEM, LOUIS BIAUT. Elected M., May 25, 1931; died Nov. 2, 1933.

FURUICHI, BARON KOI. Elected Hon. M., July 8, 1929; died Jan. 28, 1934.

MOORE, THOMAS VINCENT. Elected Assoc. M., July 12, 1926; died May 28, 1934.

NUEBLING, EMIL LOUIS. Elected M., Dec. 7, 1904; died June 5, 1934.

SPEIDEN, THEODORE. Elected M., Oct. 15, 1923; died April 17, 1934.